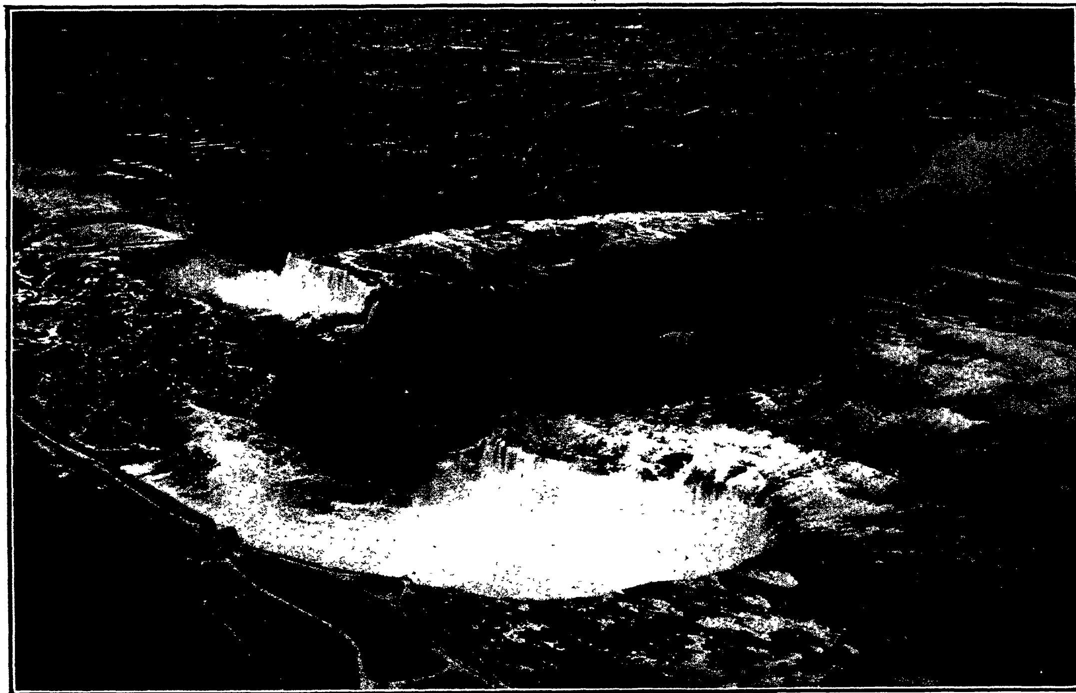
HYDRO-ELECTRIC HANDBOOK



Frontispiece

Niagara Falls, the Source of a Million Kilowatts of Generated Power.

HYDRO-ELECTRIC HANDBOOK

BY

WILLIAM P. CREAGER

AND

JOEL D. JUSTIN

With the Assistance of Nine Contributors

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BY
WILLIAM P. CREAGER AND JOEL D. JUSTIN

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PREFACE

THE aim of this book is to present a compendium of all phases of modern hydro-electric practice. It is designed to contain sufficient descriptive matter to make it valuable to the student, and also to include much that is new in theory and practice and thus to be of considerable interest to the practicing engineer.

Although the preparation of the book required several years of work, the last few months before going to press were spent in bringing the manuscript up to date. It is therefore believed that the text is truly representative of modern practice.

The editors are indebted to many engineers for helpful advice and contributions to the data contained in the book. Most of these persons have been mentioned in the text. Others to whom the editors desire to acknowledge indebtedness are Messrs. Charles A. Bissell, Albert S. Crane, Percy C. Day, Raymond D. Johnson, Frank C. Kelsey, Adolf F. Meyer, and Forrest Nagler.

Much help has been received from the engineers of the U. S. Weather Bureau, the U. S. Geological Survey, and the U. S. Reclamation Service, and from the publications issued by them; also from the members of the special committees of the various national engineering societies, particularly those of the National Electric Light Association, and from their publications.

Special credit is due to Mr. Gardner C. George for checking the text and preparing the index, to Miss May Morris for copy editing, and to Miss Dorothy Wood for work on the cut proofs.

WILLIAM P. CREAGER.
JOEL D. JUSTIN.

WATERTOWN, N. Y.,
September, 1926

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ERRATA

PAGE			
2.			Change Fig. 1 to Fig. 18 and insert opposite page 24.
4.	Line	13.	Change *BHKLMN* to *HBEKLMN*.
24.			Change Fig. 18 to Fig. 1 and insert opposite page 2.
58.	Line	29.	Change 78 to 72.4. Fig. 29 full line slightly wrong.
88.	Table	X.	Change Fig. 43 to Fig. 44 and Fig. 44 to Fig. 45.
94.	Line	13.	Change Fig. 49 to Fig. 50.
95.	Line	11.	Change 300 to 3000. Change "or" to "of."
97.	Line	7.	Change "on" to "or."
99.	Line	14.	Change *AD* to *AB*.
102.	Line	33.	Change *FGBHC* to *FBGHC*.
117.	Table	XXI.	Change 0.06 to 0.60.
134.	Line	18.	Change 76 to 78.
135.	Line	14.	Change $-$ in equation to $+$.
136.	Fig.	82.	Change d in top equation to d''.
139.	Fig.	83.	In left-hand side change h_e to h'_e, v to v' and v^2 to v'^2.
147.	Line	24.	Change (42) to (63).
148.	Eq.	67.	Change $-$ to $+$.
194.	Fig.	120.	Change n_2 and n_5 to h_2 and h_5.
201.	Line	5.	Change "force" to "face."
209.	Line	6.	Change "masonry line" to "lower nappe." See correction for page 210.
210.	Fig.	127.	This, through error, is dimensioned for lower nappe and not masonry line.
239.	Line	1.	Change Eq. (4) to Eq. (111).
295.	Fig.	193.	Main gate curve, through error, was computed for $W=0$ and is 2500 lbs. too high. Method is correct.
417.	Line	17.	Change 1913 to 1923.
641.	Line	16.	Change WR_2 to W_1
641.	Line	17.	Change "and radius" to "and square of the radius."
642.	Line	7.	Change WR^2 to Wr^2.
642.	Line	20.	Change WR^2 to Wr^2.
644.	Line	20.	Change WR^2 to Wr^2.
645.	Line	4.	Change WR^2 to Wr^2.

Meyer finds that this seems to hold true up to about 3000 ft. and then to decrease; except that, for regions where the winds are off the ocean, the precipitation increases with the altitude to greater elevations. On the opposite slopes the precipitation usually decreases at a greater rate, so that such areas have considerably less precipitation than areas at the same elevation on the side nearest the moisture supply.

Grunsky finds that, in California, the region of greatest precipitation in such cases is not ordinarily on, but rather somewhat below, the summit of the range. He points out that the variation in the annual precipitation from the western base of the Sierra Nevada, in California, across the mountains into Nevada, in the latitude of Oroville, increases from about 20 in. in the valley west of the Sierras to a maximum precipitation of about 80 in. at elevations about 1000 ft. below the crest of the range. On the plateau to the eastward of the Sierras, the precipitation drops to less than 10 in., or about one-eighth as much as at the same elevation on the western slope.

2. Rainfall Records.—The United States has been divided by the U. S. Weather Bureau into 106 climatological sections, and monthly bulletins for each section are issued by the Bureau, giving the daily and mean monthly precipitation for each gaging station in the section. Annual bulletins for each section are also published, and these tabulate the monthly and yearly means at each station for all years of record. The bulletins also contain data on temperature, wind, and humidity.

All precipitation records must be used with a full realization of their limitations, which may be summarized as follows:

(*a*) Most rainfall stations have been located at or near centers of population, which usually occur below the plane of average elevation of the section. Hence, as rainfall varies with the elevation above sea level, records obtained at these stations are not truly indicative of average precipitation.

(*b*) The monthly precipitation at stations only a few miles apart, subject to the same hydrological conditions, often differs as much as 25 per cent, owing to the limited width of the sharply defined path of severe storms, which constitute a large part of the monthly rainfall. This is particularly true during the season of thunder storms. Few districts may be found where rainfall stations are close enough together to indicate true monthly averages; but, for periods of a year or more, errors are compensating and a reasonable degree of accuracy may be expected.

(*c*) Annual precipitation varies considerably from year to year, and records of considerable length are necessary to indicate accurately the mean rainfall for a given station.

The following table, although derived by Binnie [1] for European districts, has been commonly used in this country to indicate the probable deviation of short-term records from the true mean.

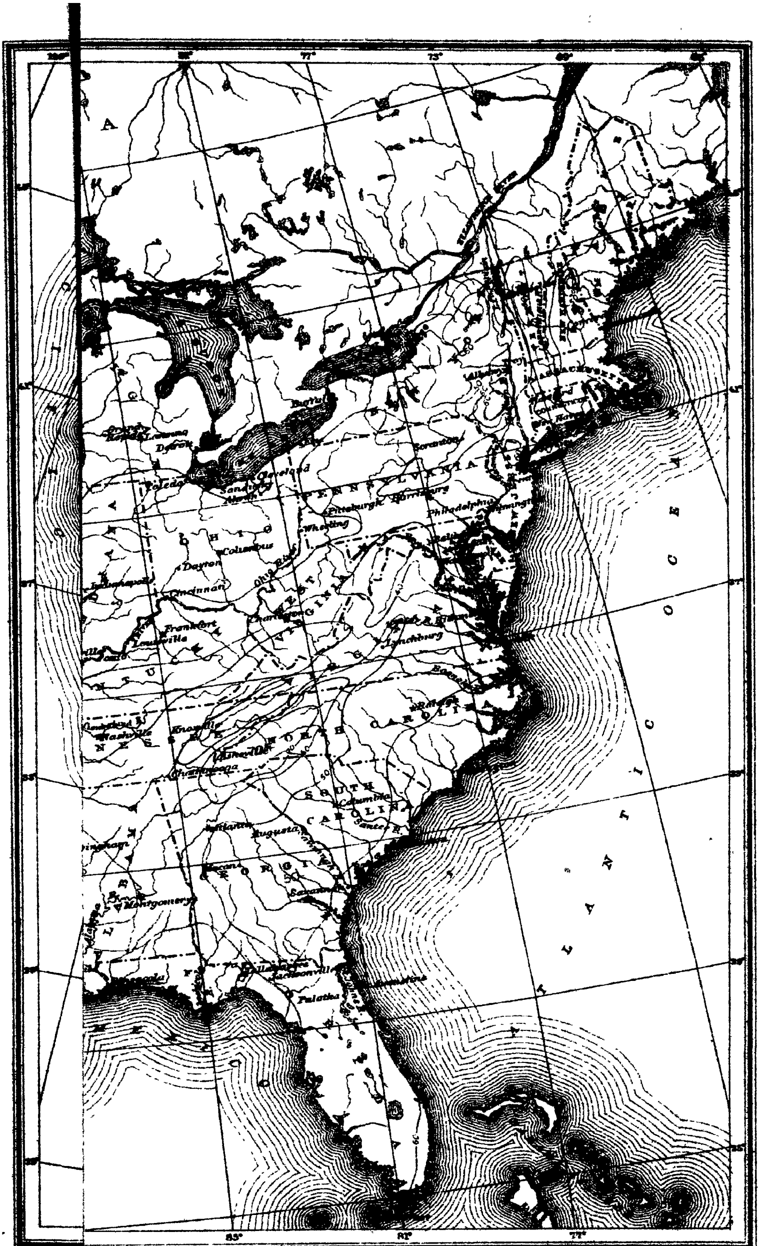
TABLE I

PROBABLE DEVIATION OF SHORT-TERM PRECIPITATION RECORDS FROM THE TRUE MEAN *

Duration of Record, in Years	Probable Percentage of Error, Plus or Minus
5	15
8	9
10	7
15	5
20	3
30	2

* Used also to indicate probable deviation of records of annual runoff.

[1] Proc. Inst. of Civil Engrs., Vol. 109, p. 89, 1892.



ECIPITATION

Courtes h in inches

Prepared by Henry Gannett
mainly from data of the
United States Geological Survey
and United States Weather Bureau

3. Mean Annual Rainfall on an Area.—In estimating the runoff of a stream having no discharge records, the recorded runoff of a neighboring stream is often used, in conjunction with a study, among other things, of the difference in mean annual rainfall on the two areas. Rainfall records on a given area are also used to determine whether the years of river-discharge records for that area represent high or low periods. The following steps are necessary in the comilpation and use of data to determine the annual rainfall over a given area or the mean for a series of years.

All annual rainfall records on, or in the vicinity of, the drainage area are tabulated, and missing records for each station approximated by interpolation between records of adjacent stations, so that the records at each station cover the same period of years. The mean annual rainfall at each station is then determined.

The use of an isohyetal map furnishes the most accurate method of determining the mean annual rainfall for a given area. An isohyetal map consists of a series of lines of equal rainfall, indicating the variation in mean annual rainfall in a manner similar to that in which the ordinary topographical maps indicate the variations in ground-surface elevations by means of contours. An isohyetal map, showing mean annual precipitation in the United States, is reproduced in Fig. 1. Isohyetals covering smaller areas would indicate variations in rainfall which it would be impossible to include on the small scale of Fig. 1. The more numerous the rainfall stations between which the isohyetals are plotted, the greater the accuracy of the map.

The maps should be drawn by plotting all rainfall stations, with their mean annual rainfall, on a topographical map of adequate scale. The isohyetals should then be drawn, due consideration being given to the effect of the topographical features on probable rainfall, because a straight-line variation of rainfall between stations does not always obtain, particularly if the two stations are in different valleys. The direction of prevailing winds and the location of the major source of moisture supply should be kept in mind as affecting the variation of precipitation with altitude. The deflection of the prevailing winds by mountain ranges and the larger valleys should be studied in connection with the resulting effect on probable rainfall.

The method of computing the mean annual rainfall, for a given area, from an isohyetal map, differs in no way from that used to determine the average elevation of an area from the contours of an ordinary topographical map.

Isohyetal maps may be used for the mean annual rainfall for the years of record or for each year of record; but, when the rainfall on an area for each year of record is required, the use of isohyetal maps is tedious and other methods are frequently employed, particularly if the available information does not permit of great accuracy.

The "weighted method" for determining the rainfall on an area is commonly used for approximate estimates and for cases where a comparison of the rainfall of various years is to be used to extend limited records of river-discharge gagings. In the latter case the relative rainfall for each year of record is as useful as the actual rainfall. The weighted method, while less exact

than the isohyetal method, embodies errors which are approximately the same for each year and therefore affects the relative rainfall very little.

In the weighted method, each rainfall station is given a weight depending upon the percentage of the whole area which it is considered to represent. R. E. Horton [2] has devised the following method for determining such weights.

Plot the different rainfall stations within and adjacent to the area on a small-scale outline map of the area, as indicated in Fig. 2. Lines connecting the different stations are drawn, as 1–2, 2–3, and perpendiculars are erected at the median points on these lines, as *AB* and *BC*. Station 1 lies nearer than any other to all points within the enclosed area *ABEFG*. Station 2

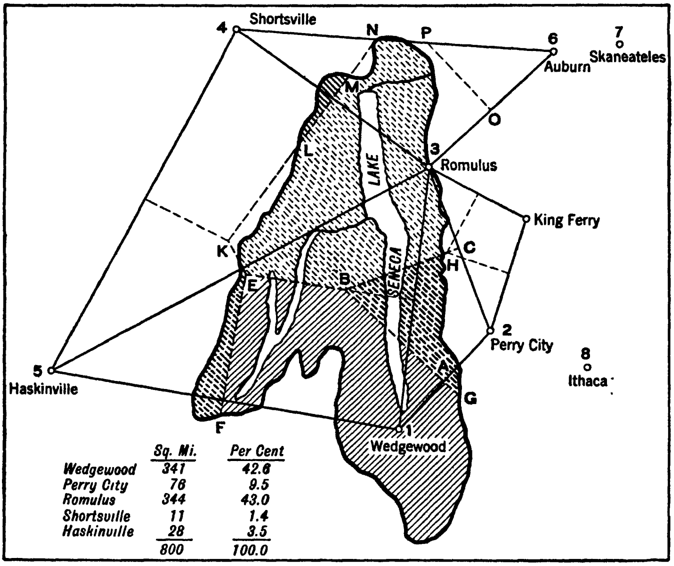


Fig. 2.

lies nearest to the portion *GABC* and Station 3 lies nearest to the portion *BHKLMN*. Points on the line *GB* are equidistant from Stations 1 and 2. Stations 6, 7 and 8 are not applicable to any portion of the area and may be eliminated. This is evident in the case of Station 6, since the perpendicular, *OP*, lies wholly outside the area. Hence no part of the area is as near to Station 6 as to Station 3. The graphical construction is easily checked by the fact that the perpendiculars to the median points of the three sides of a triangle intersect at a common point, as *K* in the case of the triangle 3–4–5. The areas of the figures nearest the different stations are determined and the corresponding proportion of the total area computed. These proportions are the weights to be given to the records at the corresponding stations.

[2] News Record, Aug. 2, 1917, p. 211.

The precipitation at each station is multiplied by its weight, and the average of the resulting weighted quantities is the probable average rainfall on the area.

As explained heretofore, it may sometimes be definitely determined that, owing to the characteristics of the topography and prevailing winds, a rainfall station will not truly represent the average rainfall on the corresponding area from which it derives its weight. In such cases, the recorded rainfall at that station for each year should be increased or decreased as the case may be; or the correction may be applied by using the actual rainfall records but changing the weight a corresponding amount, in which case the sum of the weights would not equal unity.

4. The Rainfall Year.—In making comparisons of annual rainfall to annual runoff, it should be remembered that much of the November and December rainfall may run off in January or February of the next year. A comparison of the rainfall of a calendar year with the runoff during that year would, therefore, be liable to considerable error. Since rainfall during the dry season has no appreciable effect on the total annual runoff, more accurate comparisons may be made by using a "runoff year," beginning at the end of the dry season when ground storage is about depleted.

The U. S. Geological Survey has adopted the practice of publishing runoff data in runoff years, beginning with October 1, the end of the low-stream-flow or dry season throughout most of the United States. Comparisons with rainfall should be made for a "rainfall year," covering the same period or, since runoff is not exactly coincident in time with rainfall, for large areas, a month or two ahead, or for the year beginning September 1 or August 1.

5. The Average Frequency of Dry Years.—If rainfall records of sufficient duration are available, the average frequency of occurrence of given mean annual rainfall intensities may be determined with a fair degree of accuracy, by the law of probabilities as described in Sec. 86.

The annual rainfall at Peoria, Peoria Co., Ill., has been thus examined in Table II.

The second column indicates the number of times the mean annual rainfall had a value between the corresponding amount in Col. 1 and that in the line next above. Three times during the period of fifty-three years covered by the records, the annual rainfall was between 41 and 42 in.

Column 3 is a summation of Col. 2 and indicates the number of times, during the period covered by the records that the rainfall was less than the amount indicated in Col. 1. Forty-eight times during the period, the annual rainfall was less than 42 in.

Let n = the summation of occurrences as indicated in Col. 3,
m = the total number of occurrences, in this case = 53,
p = the percentage of years in which the rainfall probably will be less than a given amount.

Then, from Sec. 86,

$$p = 100\left(\frac{n - 0.5}{m}\right).$$

TABLE II

COMPUTATIONS FOR DETERMINING THE FREQUENCY OF ANNUAL RAINFALL AT PEORIA, PEORIA CO., ILL. FIFTY-THREE YEARS OF RECORDS

(1) Annual Inches of Rainfall	(2) Number of Occurrences	(3) Summation of Occurrences, n	(4) Percentage of Years, p	(5) Yearly Frequency
24	1	1	0.944	105.8
25	1	2	2.83	35.3
26	2	4	6.60	15.15
27	2	6	10.38	9.65
28	1	7	12.27	8.15
29	3	10	17.92	5.57
30	1	11	19.8	5.05
31	6	17	31.1	3.22
32	2	19	34.9	2.86
33	6	25	46.2	2.16
34	1	26	48.1	2.08
35	2	28	51.9	1.93
36	6	34	63.2	1.58
37	1	35	65.0	1.54
38	1	36	67.0	1.49
39	3	39	72.7	1.37
40	5	44	82.1	1.22
41	1	45	84.0	1.19
42	3	48	89.7	1.11
43	1	49	91.5	1.09
44	1	50	93.5	1.06
45	....	....	....	....
46	....	....	....	....
47	....	....	....	....
48	....	....	....	....
49	1	51	95.3	1.05
50	1	52	97.2	1.03
51	....	....	....	....
52	....	....	....	....
53	....	....	....	....
54	1	53	99.1	1.01
	$m = 53$			

This equation serves to calculate the values in Col. 4, which indicates that, according to the law of probabilities, an annual rainfall less than 42 in. may be expected in 89.7 per cent of all future years.

Finally, the probable frequency, I, in years, with which the rainfall will be equal to or less than a given amount, may be found from the following equation from Sec. 86.

$$I = \frac{100y}{mp} = \frac{100}{p},$$

where y is the number of years of record, which, in this case equals m, the number of occurrences.

With this equation, Col. 5 may be calculated. The result indicates that a mean annual rainfall less than 42 in. may be expected, on an average, once in 1.11 years. Cols. 1, 4, and 5 are plotted in Fig. 3.

The curve projected through the plotted points represents the rainfall

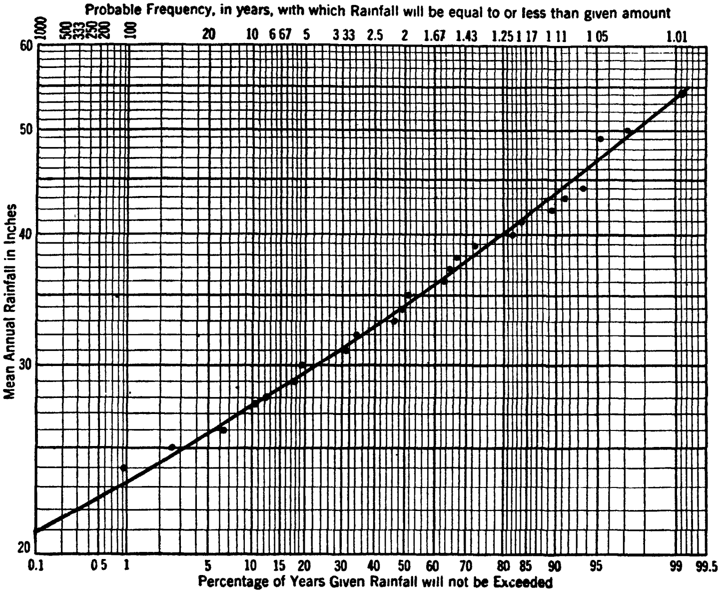


FIG. 3.—Probability Curve of Mean Annual Rainfall at Preoria, Preoria Co., Ill.

probabilities at the station as nearly as they can be determined. It indicates that a mean annual rainfall of less than 23.3 in. should be expected to occur not oftener than once in about 100 years.

This method should give accurate results for records of great length; but most records are for relatively short periods and are subject to the errors pointed out in Sec. 2. Average yearly rainfall on a drainage area may be used, instead of that of a single station, to obtain corresponding rainfall probabilities for the area.

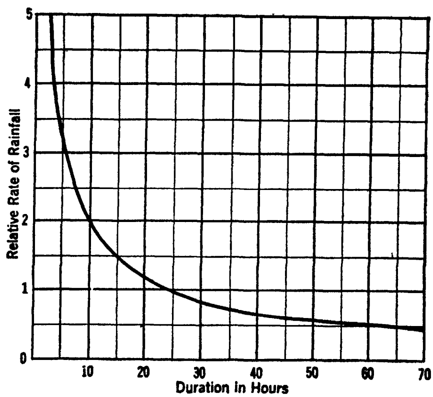


FIG. 4.—Approximate Relative Rates of Rainfall in Terms of Rate for 24 Hours. Based on Equal Frequency.

6. High Rates of Rainfall.—Studies of high rates of rainfall are made chiefly in connection with flood-flow problems. In Mead's

"Hydrology"[3] are given the equations of various authorities for high rainfall intensities. Such equations cannot be used indiscriminately. They are empirical equations, devised mostly for local conditions and therefore are not for general application. Careful studies should be made of all rainfall data in the vicinity of the project and conclusions derived therefrom. Figure 4, drawn from data compiled by Allen Hazen,[4] indicates the approximate variation of rates of rainfall with time of duration of the storm. The curve

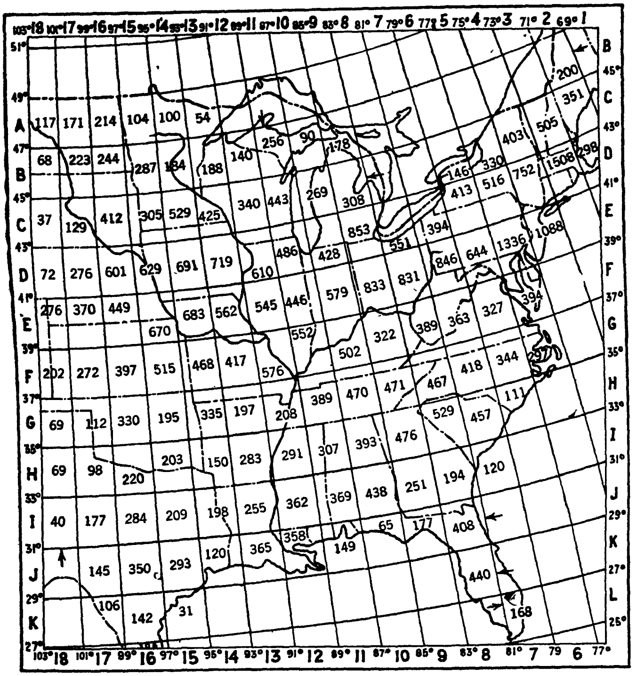


FIG. 5.—Aggregate Years of Record in Each Quadrangle. Miami Conservancy District Rainfall Studies.

shows that, for equal frequency, the average rate of rainfall during 10 hours is probably about twice that for 24 hours.

The engineers of the Miami Conservancy District, Dayton, Ohio, have compiled a multitude of data on rainfall in the Eastern United States.[5] The aggregate years of record included in their studies are given in Fig. 5. Their charts, indicating the maximum one to six-day rainfall for the years of record are reproduced herewith in Figs. 6 to 11 inclusive, and their isopluvial chart, showing intensities of twenty-four-hour rainfall of 100-year frequency, is

[3] McGraw-Hill Book Company, 1st Ed., p. 262.
[4] The Frequency of High Rates of Rainfall. Eng. News-Record, Vol. 87, p. 858, 1921.
[5] Technical Report, Part V, Miami Conservancy District, Dayton, Ohio, 1917.

reproduced in Fig. 12. As rainfall intensities vary with local conditions and may be materially different for stations only a few miles apart, Fig. 12 serves only to indicate general variations and cannot be used with accuracy for a particular locality.

Figure 13, indicating the probable frequency of rainfall of different intensities and of various periods of duration, in terms of 100-year expectations of twenty-four-hour rainfall, is based on Hazen's [6] rainfall probability studies,

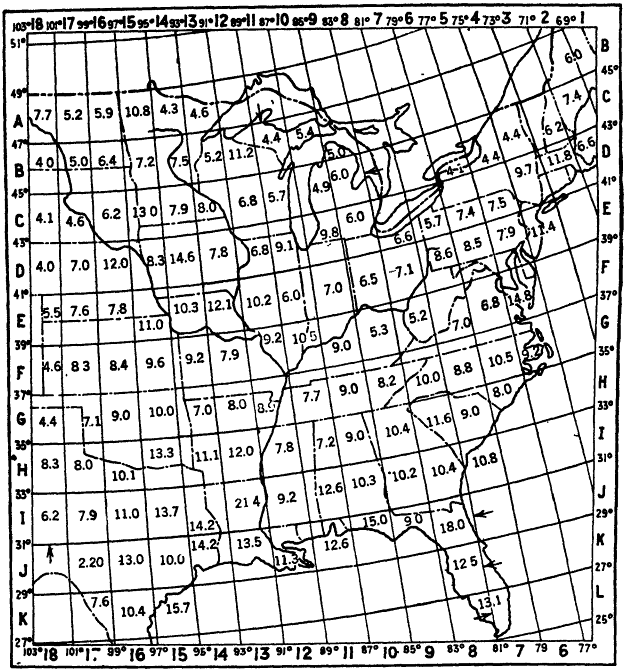


FIG. 6.—Maximum One-Day Rainfall Recorded in Each Quadrangle.

the tables of which have here been put into diagrammatic form. This figure indicates that if, for a given station, 8 in. of rainfall in twenty-four hours may be equaled or exceeded on an average of once in 100 years, then the

100-year expectancy for a 2-day storm is $1.19 \times 8 = 9.52$ in., or the
100-year expectancy for a 1-hour storm is $0.49 \times 8 = 3.92$ in., or the
500-year expectancy for a 3-day storm is $1.80 \times 8 = 14.40$ in.

The foregoing applies to rainfall at a single station. The average rainfall over a given watershed from a storm of any duration decreases as the size of the area increases. In Fig. 14, the author has endeavored to indicate the

[6] The Frequency of High Rates of Rainfall. Eng. News-Record, Vol. 87, p. 858, 1921.

result of his studies of variation of storm rainfall with area. Lack of data has prevented the publishing of conclusions that can be considered more than rough approximations. The following example will serve to indicate the application of Fig. 14. Studies of rainfall at a given station show that a rainfall of 10 in. in two days may be expected once in 100 years. It is desired to know the maximum average rainfall of the same frequency which may occur over an area of 1000 square miles surrounding that place during a two-day storm. First calculate that 10 in. in two days corresponds to an average rate of 5 in.

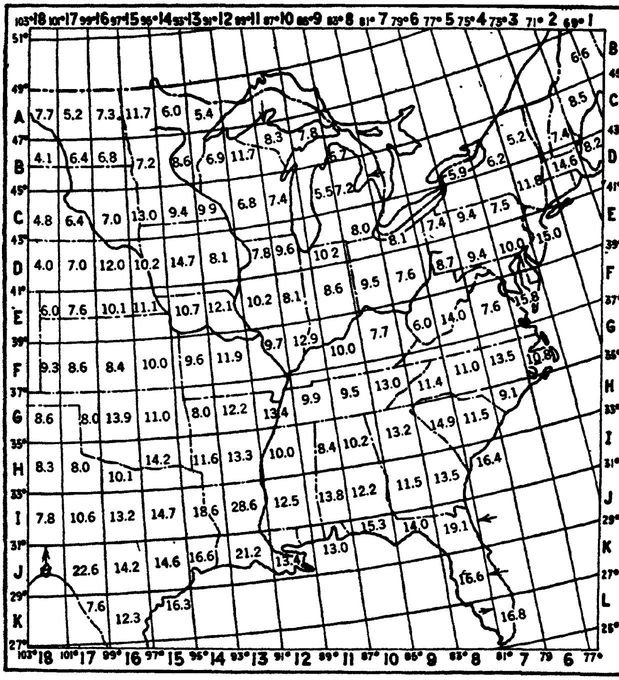


FIG. 7.—Maximum Two-Day Rainfall Recorded in Each Quadrangle.

per day. From Fig. 14, we find that the intersection of 1000 square miles with the line representing 5 in. per day is horizontally opposite 87 per cent. Therefore, 87 per cent of 10 in., or 8.7 in., is the 100-year maximum average rainfall for a two-day period over 1000 square miles.

Because of the nature of the data on which the curves are based, the 1000 square miles should have a shape corresponding to the general shape of the storm area, which is a remote possibility. Therefore there is a factor of safety in the use of the curves for flood-flow studies which is somewhat offset by the fact that the recorded maximum rainfall at a single station may not be the maximum for the storm.

It is surprising to note that available data seem to indicate that the variation of intensity with area depends upon the average rate of rainfall in inches per day, irrespective of the duration considered. Thus a storm with a peak intensity of 20 in. in four days would have the same percentage of rainfall for larger areas as a storm of 10 in. in two days, each having a rate of 5 in. per day. Existing data are too meager to permit Fig. 14 to be extended to include very intense, short-period rainfalls. Thus, a district having a maximum one-hour rainfall of 6 in., corresponding to a rate of 144 in. per day, would fall near the lower margin of the diagram, and the corresponding average rainfall over very

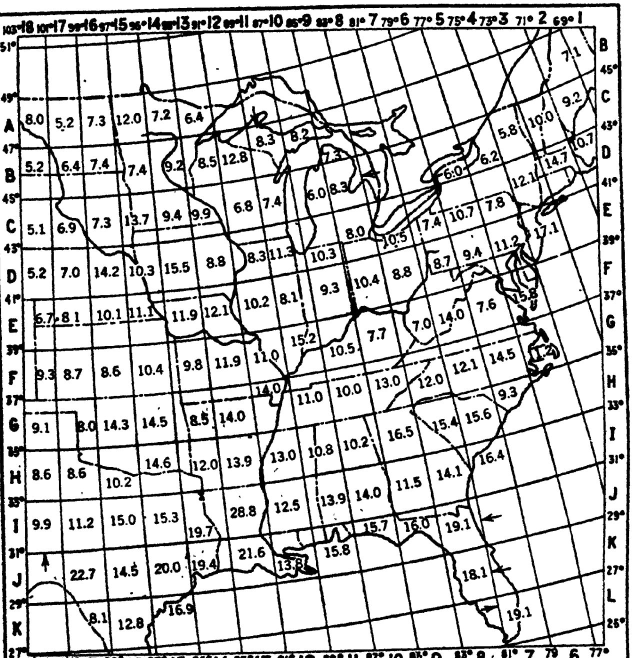


FIG. 8.—Maximum Three-Day Rainfall Recorded in Each Quadrangle.

large areas would be a very small percentage of that at the single station. Figure 14 cannot be used for areas on which there is a wide range of climatological conditions.

The larger the drainage area, the greater the duration of the storm necessary to produce flood flows, depending upon the time required for the runoff to reach the point on the river under consideration. Maximum rates of rainfall are never of long duration and, moreover, are always centered over relatively small areas. For large areas, the rainfall near the site of the dam must continue until its runoff is augmented by the flow from remote points on the

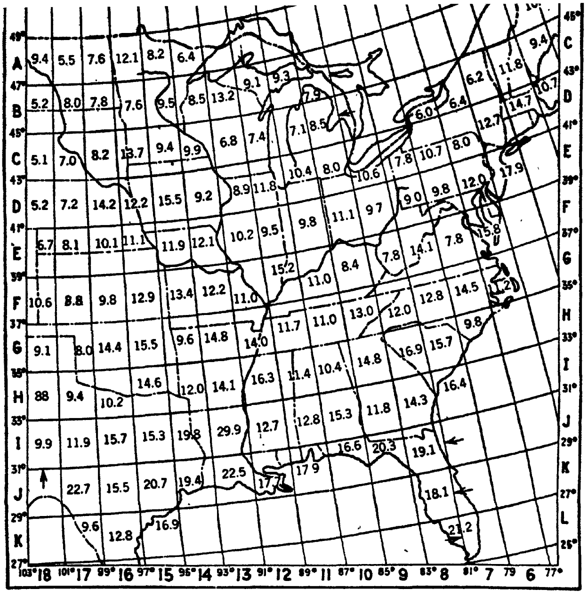


FIG. 9.—Maximum Four-Day Rainfall Recorded in Each Quadrangle.

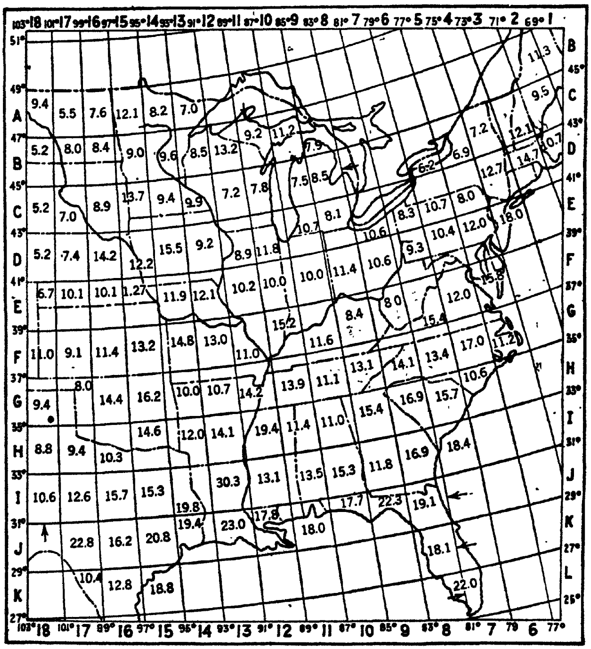


FIG. 10.—Maximum Five-Day Rainfall Recorded in Each Quadrangle.

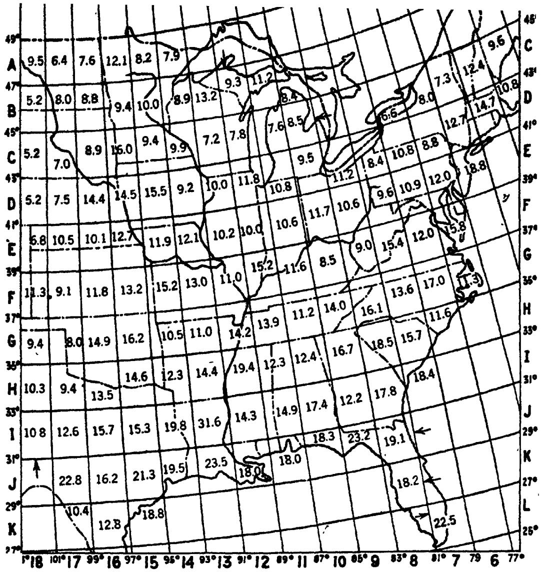


FIG. 11.—Maximum Six-Day Rainfall Recorded in Each Quadrangle.

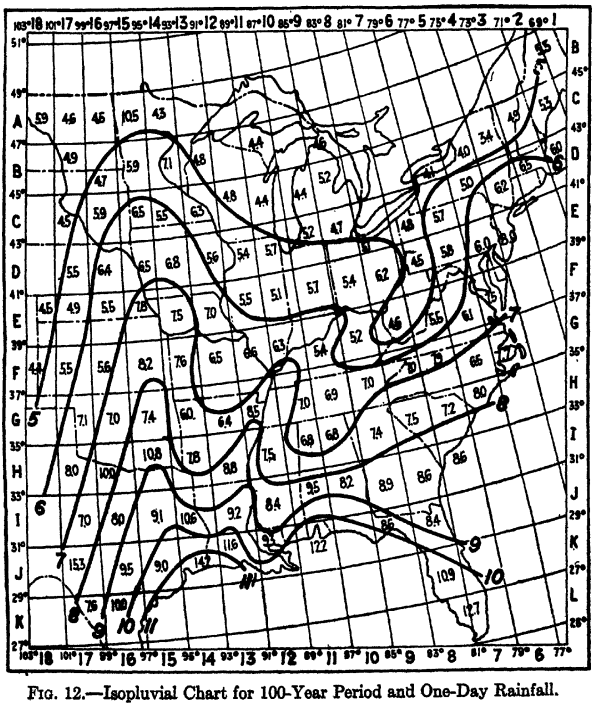


FIG. 12.—Isopluvial Chart for 100-Year Period and One-Day Rainfall.

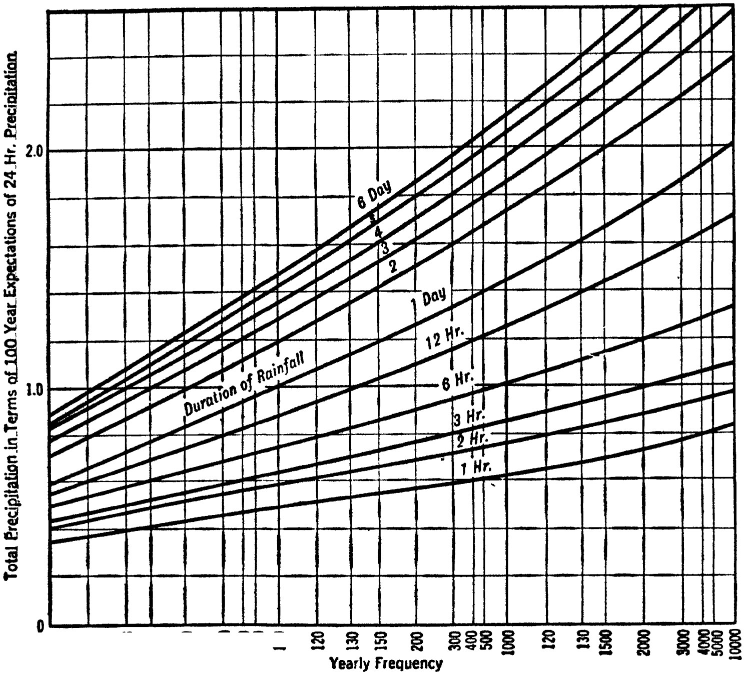


FIG. 13.—Probable Yearly Frequency Given Rainfall Will Be Equalled or Exceeded.

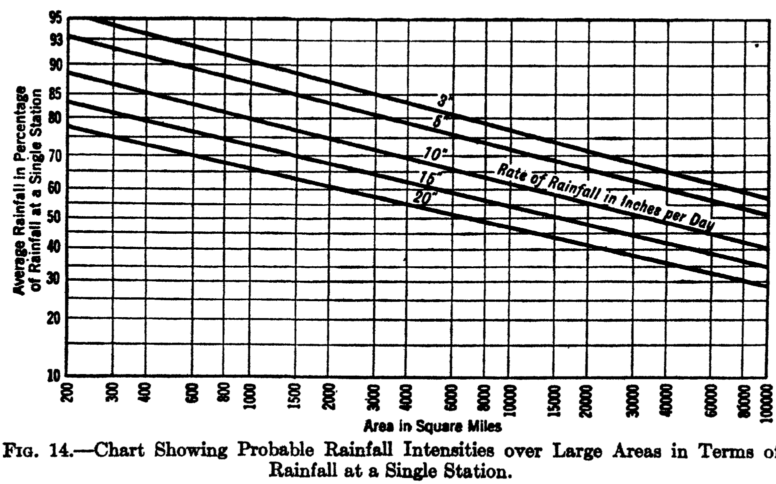


FIG. 14.—Chart Showing Probable Rainfall Intensities over Large Areas in Terms of Rainfall at a Single Station.

watershed, if a large rate of discharge is to result. Therefore, while short, intense downpours result in severe floods from small areas, they have relatively little effect on large areas, which are more sensitive to the lesser rates of rainfall over a considerable period.

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CHAPTER II

EVAPORATION

By William P. Creager

8. General.—Evaporation is by far the chief factor affecting the disposal of that part of the rainfall which does not reach the point of ultimate use. The process of evaporation consists in the changing of a liquid or solid into a vapor. Transpiration is the vaporization of water from the breathing-pores

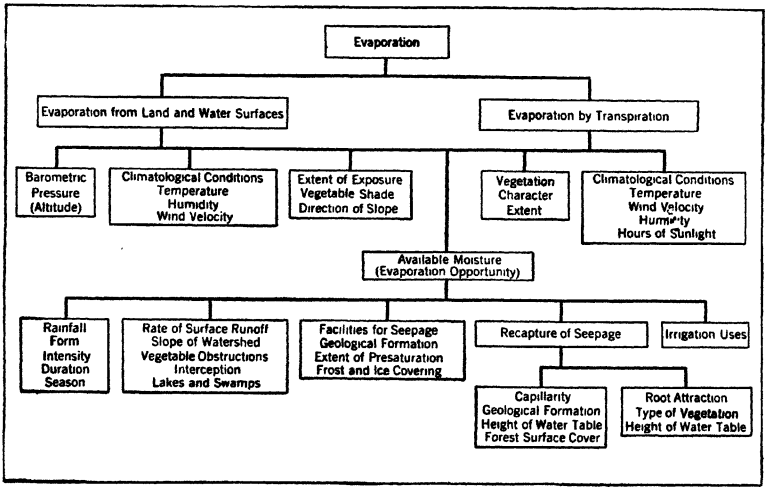

Fig. 15.—Chart Showing Conditions Affecting Evaporation.

of vegetable matter and, as such, is but a special form of evaporation. Therefore, the term "evaporation," as here and elsewhere commonly used, is meant to include all the rainfall that is returned to the atmosphere from land and water surfaces. For purposes of study, total evaporation has been divided into two groups, i.e., evaporation by transpiration and evaporation from land and water surfaces. In Fig. 15 is given a chart showing the conditions affecting total evaporation.

The numerous factors affecting evaporation, the great range and influence of each, and the almost infinite number of combinations of possible occurrences

make it impossible to estimate the distribution and amount of residual runoff solely from rainfall data. A thorough knowledge of such conditions is necessary, however, in order to extend limited records of runoff in an intelligent manner, by means of long-term annual rainfall data, and to estimate approximately, from a comparison of such factors, the runoff of a stream having no discharge records from known discharge characteristics of a neighboring stream.

Evaporation from water surfaces is dependent upon climatological conditions and, as claimed by some, upon barometric pressure as affected by altitude. Evaporation from land surfaces is subject to the additional controlling factor of extent of exposure.

Evaporation by transpiration is controlled by climatological conditions and by the character and extent of vegetation. The effect of barometric pressure on transpiration is as yet too obscure to be included in the governing conditions.

While the *rate* of evaporation at a given time is fixed by the factors mentioned, the *amount* of water evaporated during a given interval is governed also by the amount of available moisture present during that interval, or by what Horton has termed the "evaporation opportunity." The evaporation opportunity is controlled by the character of the rainfall, the rate of surface runoff, the facilities for seepage, and the possibility of recapture of seepage water by capillarity or through the roots of vegetation. Water returned to the fields for irrigation offers additional opportunity for evaporation.

9. Evaporation from Water Surfaces.—Climatological conditions affect evaporation greatly. Figure 16, from the U. S. Weather Bureau, indicates the variation of evaporation from free water surface in the United States. The lines of equal evaporation, shown in this figure, are very far from exact, as they are based on readings of dry- and wet-bulb thermometers over a limited period, supplemented by only a few actual experiments. They serve only to show the variation which may be expected to result from differences in climatological conditions.

It is known that evaporation increases directly with the average temperature and wind velocity, and decreases with the average relative humidity. It is claimed by some that evaporation increases with the altitude, although this has been seriously questioned. At any rate, the decrease in average temperature with altitude more than compensates for any increase in evaporation due to the effect of altitude. It is also known that evaporation from deep water is less than that from shallow ponds and experimental pans on land, because the average temperature of bodies of water decreases as the depth increases. However, only a few experiments have been made and, moreover, the methods of conducting such experiments have not yet been perfected. Therefore, sufficient data are not available to permit of accurate coordination of the various factors affecting evaporation from deep water or to permit writing of equations for same, although several attempts have been made.

It is often necessary to alter the estimated stream flow to compensate for evaporation from future reservoirs to be built in connection with the water power development. Usually the area of such artificial bodies of water con-

stitutes a very small percentage of the area of the watershed, and rough approximations of evaporation are well within the degree of precision in the stream measurements upon which such estimates of flow are based.

The engineers of the U. S. Reclamation Service prefer to base their estimates of evaporation from large reservoirs upon actual experiments at the site or at the nearest experimental station, rather than to use any existing equation. It is the opinion of the Climatological Division of the U. S. Weather Bureau that there is no safe method for determining evaporation save by actual observation.

Evaporation experiments at various points in the United States, compiled from U. S. Weather Bureau data, are given in Table III. Figures in italic type

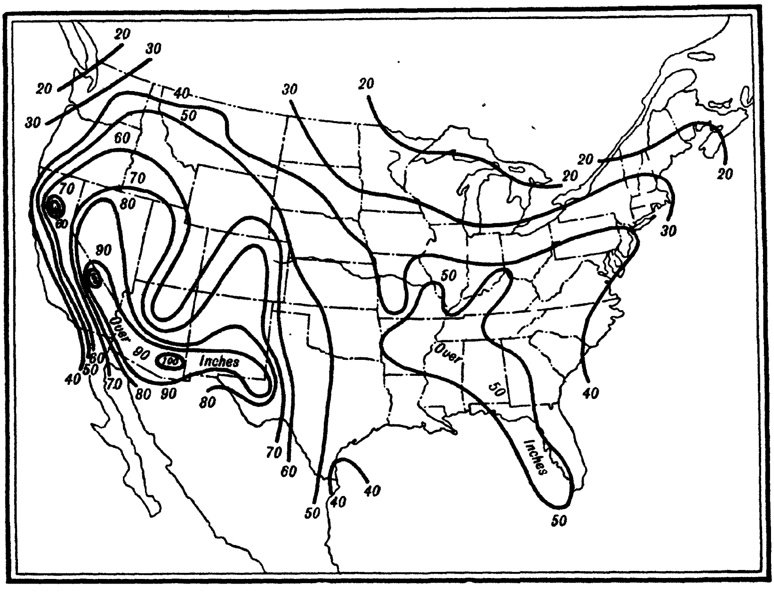

FIG. 16.—Lines of Equal Annual Depth of Evaporation in Inches from a Free Water Surface, Computed from Meteorological Observations from July, 1887, to June, 1888.

are interpolations made by the Bureau. The size of pan is given in feet. In some cases square pans were used, and in others round pans. Pans on land were located at, or within several feet of, ground surface. Floating pans were on rafts floating on the free surface of a pond or lake. Duryea and Haehl[1] conclude that evaporation from square pans of various sizes relative to a 3-ft. pan may be expressed approximately by the following relation.

2-ft. pan, 108 per cent	4-ft. pan, 93 per cent
2.5-ft. pan, 104 per cent	5-ft. pan, 86 per cent
3-ft. pan, 100 per cent	6-ft. pan, 80 per cent

[1] See Reference No. 6, Sec. 16.

This relation is substantially that of the U. S. Weather Bureau.[2] This Bureau also concludes that the evaporation from floating pans equals 80 per cent of that from land pans of the same size, and that evaporation from large reservoirs is equal to about 62 per cent of that from 3-ft. floating pans. In all probability, the error involved in the comparison between square and round pans is well within the accuracy of the relation given.

10. Evaporation from Land Surfaces and Transpiration.—Evaporation from land surfaces and transpiration are governed by the same general climatological factors as evaporation from water surfaces; but evaporation decreases with the extent of vegetable shade and transpiration increases with the hours of sunlight during the growing season. Both are greater on watersheds sloping to the south and subject to the direct force of prevailing winds.

The character, condition, and extent of vegetation have a direct bearing on the rates of evaporation and transpiration; but the relation is not well established. The author's careful examination of the published results of many experiments and of the opinions of numbers of writers has led to the conclusion that no general agreement has been reached. Mead has said [3] that " Observations in Wisconsin indicate that little change occurs in the flow of streams after deforestation . . . ; but that about the same amount of water is vaporized by the second growth and the crops or other vegetation on deforested areas."

It would seem, therefore, that at the present time we must assume the preponderating influence to be that of temperature and evaporation opportunity, the latter being affected by the nature of the geological and topographical features of the watershed.

Vermuele's [4] tabulation of mean annual temperature and corresponding mean annual evaporation, for a number of streams in this country and England, has been plotted in Fig. 17. There seems to be clearly defined evidence that evaporation generally increases with the temperature. It is seldom, however, that the mean temperatures of two districts, near enough to be comparable for stream-flow studies, are materially different except in mountainous regions.

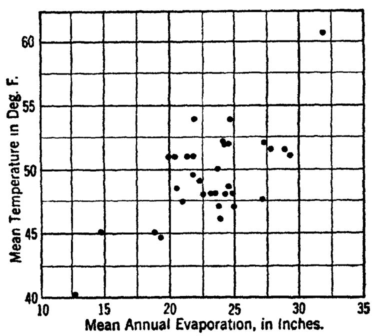

FIG. 17.—Relation between Temperature and Evaporation for Different Streams.

11. Effect of Character of Rainfall on Evaporation Opportunity.—The percentage of evaporation from short, light showers is quite excessive, particularly if followed by sunshine. Heavy rains run off quickly, while light,

[2] See Reference No. 10, Sec. 16.

[3] The Flow of Streams and the Factors that Modify It, by D. W. Mead. University of Wisconsin Bul. 425.

[4] Annual Report of the Geological Survey of New Jersey; Report on Forests, Part I, by C. C. Vermeule, 1899.

TABLE III

Evaporation Experiments in the United States

Station	Years of Record	Location of Pan	Diameter of Pan	Evaporation in Inches												
				Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	The Year
Birmingham, Ala.	1	Floating	4	*1.50*	*1.50*	*2.25*	4.45	5.91	7.28	7.36	7.34	6.00	*4.00*	*2.25*	*1.50*	51.34
Silverhill, Ala.	3	Land	4	2.68	3.13	4.80	5.68	6.06	7.38	6.48	6.16	6.16	4.99	6.78	2.30	62.60
Granite Reef, Ariz.	1	Land	4	4.59	*4.75*	*6.25*	*9.00*	*11.50*	*13.50*	*14.25*	14.23	13.76	11.31	7.39	4.65	115.18
	1	Floating	4	*4.25*	*4.40*	*5.25*	*7.00*	*9.50*	*12.00*	12.75	12.50	11.00	8.31	6.56	4.22	97.74
Mesa, Ariz.	5	Land	4	2.59	3.44	5.85	8.45	10.47	11.44	10.53	8.56	7.00	4.96	3.34	2.82	79.45
Roosevelt, Ariz.	6	Land	4	2.37	3.06	5.55	8.06	11.04	13.53	12.45	10.50	8.76	5.88	3.59	2.62	87.41
Willcox, Ariz.	5	Land	4	3.28	4.48	7.69	10.05	11.36	11.78	10.38	8.70	7.56	6.57	4.60	3.53	89.98
Yuma, Ariz.	5	Land	4	3.22	4.12	5.94	7.84	8.27	9.01	10.48	9.58	7.10	4.98	3.17	2.88	76.59
Salton Sea, Cal.	1	Land	2	5.08	7.42	*12.50*	*15.75*	*19.00*	*21.50*	22.15	18.50	15.50	13.19	7.49	6.42	164.50
	1	*	4	3.61	5.01	*6.75*	*9.00*	*11.00*	*13.50*	14.77	12.53	12.40	9.20	6.21	4.67	108.65
Indio, Cal.	1	Land	6	3.18	5.08	7.50	12.05	15.84	16.11	16.34	13.78	12.37	8.91	5.17	3.00	119.33
Mecca, Cal.	1	Land	6	2.92	5.00	8.07	10.87	12.72	14.23	15.21	13.22	10.29	8.17	4.13	2.98	107.81
Browley, Cal.	1	Land	6	3.05	5.00	8.00	10.47	13.79	13.68	14.14	11.26	10.15	6.99	4.09	2.66	103.55
Mammoth, Cal.	1	Land	6	4.24	5.67	8.99	12.02	15.52	16.75	18.00	13.73	12.16	9.49	5.26	3.70	125.53
Chula Vista, Cal.	3	Land	4	3.16	3.15	4.91	6.10	6.00	7.13	7.41	6.95	6.08	4.84	3.70	2.96	62.39
Dodgeland, Cal.	3	Land	4	1.16	1.77	2.90	5.79	8.38	10.30	11.42	8.80	6.23	3.97	1.78	0.95	63.45
Oakdale, Cal.	4	Land	4	1.45	2.36	3.06	5.98	9.44	13.44	15.17	13.92	8.92	5.47	2.98	1.12	83.31
Tahoe, Cal.	6	Floating	4	1.74	.....	.....	3.01	3.69	4.71	5.94	6.27	5.00	3.42	2.67	1.70	
Deer Flat, Idaho	1	Land	3	*1.50*	*2.25*	*4.00*	*7.25*	10.68	11.05	11.15	11.77	*9.75*	5.40	2.70	*1.50*	79.00
	1	Floating	4	*2.00*	*2.75*	*4.25*	*6.00*	7.90	9.59	10.59	12.16	*9.25*	5.42	5.52	*2.00*	77.43
Jerome, Idaho	3	Land	4	.....	.....	2.65	4.14	6.24	7.92	8.99	7.03	3.56	1.78	1.09		
Tribune, Kan.	3	Land	4	.....	.....	.....	6.50	9.68	7.22	13.02	10.78	8.30	6.38			
Wichita, Kan.	3.5	Land	4	.....	.....	3.46	6.72	6.49	7.39	9.50	8.68	7.67	5.94	2.36		
Gardiner, Me.	5.5	Land	4	.....	.....	.....	2.45	4.49	5.21	5.35	4.66	3.50	2.18	0.99		
Centerville, Minn.	2.5	Land	4	.....	.....	.....	.....	5.84	6.60	8.92	6.29	4.51	2.45			

Columbia, Mo.	6	Land	4	.....	.....	2.19	3.86	5.26	6.68	8.39	6.62	4.35	3.04	1.58		
	3	Land	4	.....	.....	2.88	4.89	5.28	7.11	8.63	6.72	5.19	3.46	2.16		
Dutch Flats, Neb.	1	Land	4	*1.75*	*1.75*	*3.00*	*4.50*	*6.25*	8.05	10.95	9.39	7.44	5.59	*4.00*	*3.00*	65.67
Lincoln, Neb.	5	Land	4	.....	.....	.....	5.55	6.80	8.54	9.72	8.66	6.69	4.75			
North Platte, Neb.	3	Land	4	.....	.....	.....	5.18	5.72	6.73	12.56	7.25	5.50				
Pahrump, Nev.	2	Land	4	2.29	2.89	5.64	8.60	10.38	11.70	12.12	10.65	8.67	5.22	2.54	1.92	82.62
Fallon, Nev.	1	Floating	4	*1.75*	*1.75*	*2.25*	*3.25*	*5.25*	7.86	9.86	8.70	5.13	3.35	*2.50*	*2.00*	53.65
Elephant Butte, N. Mex.	6	Land	....	2.44	4.41	8.32	10.98	14.29	14.10	12.25	10.74	9.31	8.14	4.45	3.03	102.46
	1	Land	4	*2.50*	*2.75*	*4.50*	*8.00*	*11.50*	13.45	11.57	10.48	8.58	6.76	3.86	*3.00*	86.95
Carlsbad, N. Mex.	1	Land	4	*5.00*	*5.50*	8.94	11.68	12.86	12.40	12.00	11.03	9.76	7.58	5.50	*5.00*	107.25
Lake Avalon, N. Mex.	1	Floating	4	*4.50*	*4.50*	5.51	7.45	10.12	11.05	12.88	*12.00*	*9.50*	*7.00*	*5.75*	*4.50*	94.51
Agricultural College, N. Mex.	3	Land	4	2.89	4.27	7.93	10.17	11.96	12.49	12.11	10.32	8.67	6.21	3.73	2.80	93.55
Sante Fé, N. Mex.	5.5	Land	4	1.76	2.12	4.11	6.17	8.62	10.16	8.60	7.62	6.46	4.68	2.71	1.72	64.73
Tucumcari, N. Mex.	2	Land	4	.....	.....	.....	7.81	8.26	7.66	9.73	9.74	7.39	5.11	2.32		
Albany, N. Y.	2	Land	4	.....	.....	.....	.....	4.92	5.42	5.12	5.06	3.84	2.55			
Ithaca, N. Y.	2	Land	4	.....	.....	.....	2.18	4.76	5.34	5.30	4.88	4.08	2.72			
Chapel Hill, N. C.	1	Land	4	.....	.....	.....	3.69	3.32	4.59	5.15	5.45	4.47	3.20	1.49	1.28	
Ohio St. University, Ohio	4	Land	4	.....	.....	.....	.....	4.53	6.02	6.59	5.21	4.31	2.40	0.97		
Wooster, Ohio	5.5	Land	4	.....	.....	.....	3.05	4.46	5.66	6.24	5.38	3.64	2.29			
Hermiston, Ore.	1	Floating	4	*1.25*	*1.25*	*3.00*	7.28	7.89	9.54	12.04	11.07	7.35	3.88	*2.00*	*1.50*	68.05
	1	Land	3	*1.50*	*1.75*	*4.25*	9.28	11.38	13.84	17.48	16.89	10.09	6.08	*3.00*	*1.75*	97.29
Rapid City, S. D.	6	Land	4	.....	.....	.....	3.63	5.48	6.92	8.29	7.38	5.22	3.19			
Austin, Tex.	6	Land	4	2.69	3.24	5.42	6.57	6.77	8.05	8.74	8.50	6.63	5.10	3.21	2.72	67.64
Myton, Utah	4	Land	4	.....	.....	.....	5.70	9.57	9.10	9.84	8.48	6.42	3.58	0.41		
Pinto Dam, Utah	4	Land	4	.....	.....	.....	4.84	9.45	12.26	11.36	9.59	8.01	4.98	2.34		
Pravo, Utah	4	Land	4	.....	.....	3.24	3.55	6.18	7.03	7.21	6.10	4.09	2.08	0.58		
Kachess Lake, Wash.	3.5	Land	4	.....	.....	.....	.....	3.33	5.14	5.99	4.41	2.12				
Walla Walla, Wash.	6	Land	4	.....	.....	2.57	3.79	5.18	7.37	9.57	8.75	4.89	3.23			

* Suspended 2′ above water surface.

continuous precipitation is conducive to percolation. As temperature, more than any other factor, controls evaporation, winter precipitation produces the greatest percentage of runoff. The amount of evaporation increases with the annual rainfall, but the percentage of evaporation decreases with the rainfall.

Snowfall and frozen rain remain for long periods subject to evaporation, but they occur during seasons when evaporation is relatively small. This form of precipitation, therefore, is favorable to large runoff.

12. Rate of Surface Runoff.—Evaporation varies inversely with the rate of surface runoff, or directly with the length of time surface water is subjected to the influences governing evaporation. The average slope of the watershed is the principal factor governing the rate at which the surface water passes to the streams.

All plant growth affects, to some extent, the rate of surface flow. Forest cover is considered by many to reduce flood flows because of its offering great obstruction to surface flow. In all probability, the dense underbrush accompanying forest growth has the greater influence. However, this effect may be reversed if forest shade delays the melting of snow until the heavy spring rains set in. The relatively greater effect of forests on delayed runoff is probably negligible during normal rains.

For each rainfall, practically a constant amount of water is intercepted by vegetation and later is evaporated without reaching the ground. The amount would naturally be greater for denser growths, but the proportion of the rainfall thus held back is small except for very light rains.

Lakes and swamps, because of their great retarding effect on the rate of surface runoff, afford the greatest opportunity for evaporation. In some climates the evaporation from such areas may greatly exceed the rainfall.

13. Facilities for Seepage.—Evaporation varies inversely with the porosity of the soil. Sandy soils greatly increase the percolation and hence the low-season runoff, and tend, to a great extent, to equalize the flow of streams.[5] The depth of the pervious layer of soil is as important as the porosity of the soil; for, unless the water seeps to depths beyond reach of capillarity and the attraction of roots, much will be returned to the surface and evaporated. Clayey soils and rocky slopes are not conducive to seepage. Plowed ground is one of the greatest agents of percolation, because, by the action of the plow, the hard top layer of the soil is loosened and its capacity for absorbing water is increased. Plowed fields should therefore increase the low-season flow by ground storage, provided the depth of soil is sufficient to conduct the water beyond reach of crops.

Flat slopes, lakes, swamps, and all vegetation retard flow and promote seepage, but, at the same time, provide greater evaporation opportunity unless the soil is quite pervious. Such agencies, therefore, tend to promote regularity of stream flow but reduce the total runoff.

Ground that is presaturated by long, heavy rains suffers a material reduction in capacity for percolation. This is one of the principal contributing causes of excessive floods. Frozen, and particularly ice-covered ground, forms an efficient barrier to seepage, as a result, many winter rains in northern lati-

[5] See Sec. 17.

tudes appear entirely as stream flow and are, indeed, sometimes augmented by melting snow.

14. Recapture of Seepage.—A considerable portion of the rainfall, having disappeared as seepage, is returned to the surface by capillarity and through the roots of vegetation. Percolation is resisted by capillarity, which tends continually to supply the surface of the soil with moisture from the water-bearing strata. According to Hazen, the height to which water will be lifted by the capillarity of the soil varies inversely as the square of the size of the individual grains. Evaporation, through recapture, therefore, varies inversely with the size of the soil grains and the depth of the water-bearing soils. The influence of capillarity is said to extend, in some soils, to a depth of 30 ft.

Clayey soils not only resist percolation by their relative imperviousness, but also have greater capillary attraction. Hence, such soils are not productive of runoff by seepage. Sandy soils, on the other hand, promote seepage and resist capillarity and produce well-regulated high-percentage runoff.

The major portion of the water used in transpiration is derived through the root systems from the soil. Deep root systems have therefore the greatest capacity for recapture. It is for this reason that heavy forest growth is considered to have a relatively unfavorable effect on the extent of permanent seepage during the growing season. This is true only if the water table is relatively low, because, for high water tables, an equal opportunity for recapture by transpiration exists for the smaller root systems.

15. Irrigation Uses.—The amount of water used for irrigation and the percentage which returns to the stream depend upon the nature of the crops, the surface and subsoil, the amount of water available, and the irrigation methods in use. Unless detailed knowledge of the existing or proposed irrigation project is available, it is better to assume that at least two-thirds of the water used for irrigation is lost, although tests have indicated that 65 per cent may be returned to the stream.[6]

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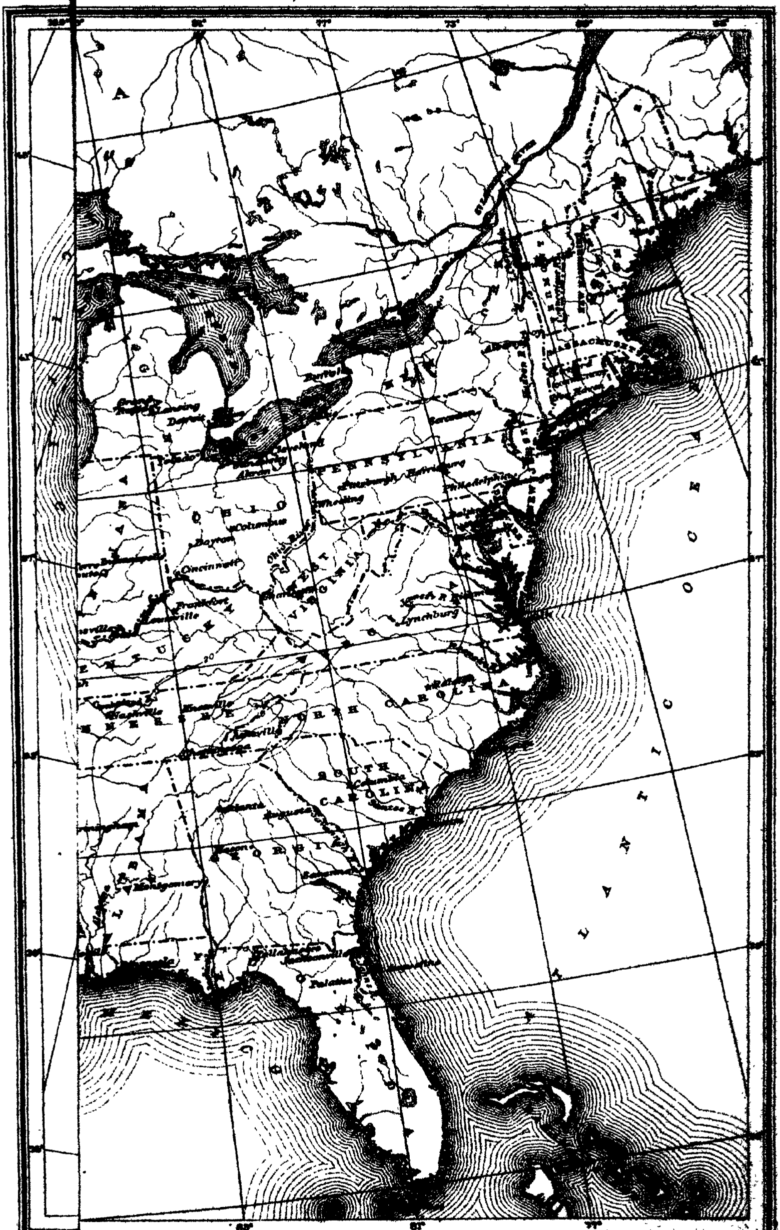
[7] See References 7 and 10.



L RUN-OFF

Cou in inches

Prepared by Henry Gannett
ly from data of the
United States Geological Survey

CHAPTER III

FACTORS AFFECTING RUNOFF

By William P. Creager

17. General.—A knowledge of the factors affecting the runoff characteristics of drainage areas is particularly essential when, in the absence of streamflow records, the recorded runoff of a neighboring stream is used to estimate the probable amount and distribution of runoff of a stream to be developed for power, by a comparison of the relative runoff characteristics of the two watersheds. This chapter, therefore, treats of the practical application of this knowledge to such comparisons.

The disposition of rainfall, the sole source of runoff, is described in Sec. 1. The percentage of rainfall that appears as runoff at the site of proposed use varies widely. Figure 18 indicates the average annual runoff in the United States. The runoff from individual areas differs considerably from the averages shown, depending upon local conditions.

For streams without complete artificial storage, the distribution of the total runoff becomes of paramount interest as affecting the water available for power.

The extent of ground storage, or the ability of the ground to absorb a part of the rainfall, to be delivered to the stream weeks or months later in the form of seepage, governs to the greatest extent the distribution of runoff. Most of the flow of streams during periods of drought is derived from ground storage.

Evaporation is by far the most important factor influencing the percentage of rainfall that finds its way to the stream. The conditions affecting evaporation are indicated in Fig. 15 and described in Chapter II. The rate of evaporation is governed principally by temperature; but the amount of evaporation and its seasonal distribution, in which the engineer is most interested, are controlled principally by the geological formation of the watershed.

Diversion of water from the drainage area is sometimes made for municipal and industrial water supplies, for other water powers, for irrigation, or for other purposes. Such amounts can usually be closely determined and deducted from the computed runoff if they occur above the point of proposed use.

18. Deep Seepage.—Deep seepage is that portion of the runoff which passes through the underlying earth or rock strata below any possible intercepting cut-off at the dam, or which seeps through the divide into another watershed. It is usually inconsequential; but cases have been cited where seepage from a very small drainage area into an adjacent drainage basin has caused serious losses.

For small drainage areas, a careful comparison of the geological formation with that in adjacent valleys should always be made, to determine the possibility of deep seepage.

Figure 19 is a section through two watersheds and shows how deep seepage may augment the flow of one stream to the detriment of another. The percentage of rainfall lost by deep seepage into adjacent watersheds becomes rapidly less as the drainage area increases.

In some sections of this country, the geological formations are such that large subterranean cavities exist in the bed rock and receive part and sometimes all of the flow of the stream. This is a special form of deep seepage.

19. Geological Characteristics of the Watershed.—Steep, impervious areas are productive of a large percentage of total runoff, but are without adequate ground storage and hence are characteristic of small dry-season flows. Hydrographs of such areas show flashy characteristics. Rock surfaces are not necessarily impervious, although generally so. Some classes of rock on relatively flat surfaces are quite pervious and offer excellent opportunity

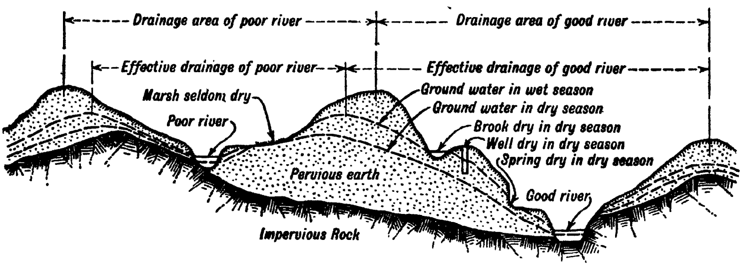


FIG. 19.—Wet and Dry Season Ground Water Stages, Showing Deep Seepage Loss from a Watershed.

for ground storage. Clayey surfaces and other shallow earth covering over impervious rock absorb little water.

The large cavities which exist in most limestone formations are excellent storage reservoirs of ground water if the earth covering is pervious. Shales and shaley sandstone are quite impervious.

Table IV, taken from the 1915 Report of the Water Supply Commission of Pennsylvania, indicates the extreme divergence of flow, during times of drought, from adjacent drainage areas.

Spring and Bold Eagle Creeks, in Center Co., Pa., have approximately equal drainage areas. The remarkably better yield from Spring Creek is attributed entirely to the greater opportunities for ground storage afforded by the limestone formation which prevails generally throughout its drainage area. The better storage of Spring Creek is also indicated clearly by the uniform December and January flow, as compared with the increase from 8 to 68 per cent runoff for the same months from Bald Eagle Creek following a wet period which embraced both areas.

Flat, impervious areas are productive of small percentages of runoff at all seasons. Stream flow from such areas is far from uniform, but not as flashy

TABLE IV

COMPARISON OF LIMESTONE AND NON-LIMESTONE YIELDS DURING LOW PERIOD OF 1914–1915

Month	Spring Creek (145 Sq. Mi.), Limestone Formation		Bold Eagle Creek (140 Sq. Mi.), Non-limestone Formation	
	Discharge, Sec. Ft. per Sq. Mi.	Per Cent of Runoff to Rainfall	Discharge, Sec. Ft. per Sq. Mi.	Per Cent of Runoff to Rainfall
September	1.23	971	0.27	14
October	1.24	60	0.04	2
November	1.30	118	0.10	9
December	1.26	45	0.22	8
January	1.70	43	2.69	68

as that from steep, impervious areas. Flat areas, having coarse, sandy soil of considerable depth, produce the most regular runoff.

The relative imperviousness of watersheds can be approximated by a careful inspection of the surface, in which consideration should be given to the extent of out-cropping rock and of the subsurface conditions at ravines and excavations.

20. Extent of Lakes and Swamps.—Increased discharge from a natural body of water is produced by a rise of water surface, which results in a greater depth and slope of the outlet channel. This necessary rise of water surface provides temporary storage which retards the flow of the stream. The extent to which the flow will be equalized depends upon the extent of rise and the area of the water surface. Narrow outlets, requiring large fluctuations in stage for changes in stream discharge, are indicative of good natural storage and relatively steady runoff characteristics.

Although the tendency of all lakes and swamps is to equalize the flow, evaporation from the surface of water reduces the total runoff. The effect of evaporation is negligible during freshets and floods, and the maximum rate of runoff is materially reduced with the extent of lake and swamp area. During extremely dry periods, however, excessive evaporation may equal or exceed the surface flow and result in zero runoff if the area of water surface is a large percentage of the total tributary basin.

The percentage of runoff that is evaporated from water surfaces depends upon the proportion of area of water to land surface and upon the rate of evaporation from the water surface. It was indicated in Sec. 9 that evaporation from water surfaces decreases as the depth of water increases. Deep lakes are, therefore, productive of better regulation than shallow ponds and swamps. The extreme uniformity of flow of the Richelieu River, shown in Fig. 48, is caused by the equalizing effect of Lake Champlain, which has considerable depth. Shallow lakes with wide outlets, and particularly swamp and marsh areas, while ordinarily beneficial to the regulation of ordinary and flood flows, are a menace to dry-season flows on account of extreme evaporation.

The equalizing effect of natural bodies of water during seasons of low flow depends principally, therefore, upon two conditions: first, the evaporation opportunity, which is fixed by the area of water surface in proportion to the tributary area and the depth of water; and second, the storage characteristics, which are dependent upon the absolute surface area and the width of outlet. The effect of these conditions may be beneficial to good regulation or decidedly the reverse, depending upon whether evaporation or storage has the greater influence.

The effect of lake and swamp area should be evaluated according to whether the rate of minimum flow or that of ordinary flow has the greater influence on the value of the development. The revenue from the output of developments, without auxiliary plant and serving a primary market, is fixed by the rate of minimum runoff and is therefore affected adversely by great extents of shallow lakes and swamp areas. Developments with auxiliaries, or serving a secondary power market, or having limited storage, depend upon the rate of ordinary flows and may be benefited by shallow lakes and swamps. Developments operating in conjunction with large storage reservoirs need little or no assistance from natural storage, and hence the output is reduced by shallow lakes and swamps in proportion to their effect on the total runoff.

21. Vegetation.—The influence of the extent and nature of vegetation on the amount and distribution of runoff is felt principally through its effect on evaporation. It seems to have been proved conclusively that cultivation of pervious soils overlying deep, impervious strata promotes underground storage and hence uniform flow. Meyer says that, if the level of saturation is about 20 ft. below the surface in clay subsoils, and about 10 ft. in sandy subsoils, the ground water is safe from all vegetation except matured forest cover.

The relative effect of cultivation and of forests on low-season runoff is much the same. Tillage of the ground for cultivation opens and roughens the hard, smooth surface and promotes seepage. Roots of the forest trees open pathways for seepage, not only in the ground, but also in bed rock by getting into and opening seams, and the humus cover of virgin forests holds back the flow and increases the opportunity for seepage.

Transpiration is greater in cultivated than in forest areas during periods of high ground water, and less during other periods. This is because the transpiration demands are greater for cultivated areas for equal ground supply; but the roots of forest trees extend to a greater depth.

Deforestation of steep, rocky slopes has a great tendency to decrease low flows. Flat areas having thick, sandy soil are probably benefited by deforestation because of the removal of the transpiration demands of deep root systems, while, at the same time, assistance in holding back flow to promote seepage into the soil is less needed.

The demands of all forms of vegetation on ground water are excessive during drought, and, when an impervious under stratum holds the water table close to the surface, reduction of ground storage by transpiration is excessive, particularly for cultivated areas. On the other hand, cultivated areas with deep ground-water systems are good low-season flow producers, and cultivation of such areas is particularly beneficial if the slopes are steep.

The effect of vegetation on low-water flow is relatively small in comparison with the influence of the geological and topographical characteristics of the watershed.

22. Geographical Features.—The dry-season runoff per square mile is considered to increase with the area of the watershed; that is, the natural regulation for large areas is better than for small ones. Figure 20, plotted from data contained in the Geological Survey of New Jersey, 1894, indicates this tendency; but the relation is not well established for the territory covered by the data. For small, rocky watersheds and for small areas subject to deep seepage, the low-water flow may be negligible; but, for the streams plotted in Fig. 20, the variation in low runoff per square mile is probably caused by greater frequency of rainfall in the larger areas. While minimum flows per square mile increase with the area, flood flows per square mile decrease as the area increases. It would, therefore, seem that the effect of the size of drainage area on ordinary flows and total runoff is less than for minimum and maximum discharges.

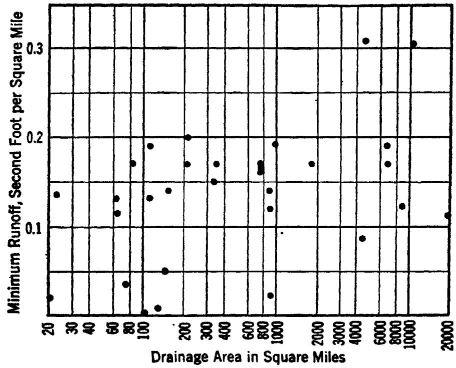


FIG. 20.—Comparison of Minimum Flow with Drainage Area. North Atlantic Coast Streams.

Other things being equal, evaporation is less for higher elevations. Moreover, under certain conditions explained in Sec. 1, rainfall increases with the elevation above sea level. Therefore, total runoff per square mile of watershed may be expected to be greater for the higher elevations. Hazen [1] has found that a rough approximation for inches of runoff from land surfaces in Massachusetts may be expressed by the following equation:

$$R = 19.4 + 0.0064E,$$

where E is the average elevation, in feet, of the watershed above sea level. From this equation it would seem that the total inches of runoff at an elevation of 1000 ft. above sea level would be 33 per cent greater than at sea level. The relation applies to land areas only, and was deduced by correcting the observed total runoff on the basis that 3 square miles of water surface, on account of greater evaporation, produce as much runoff as 1 square mile of land area.

Higher watersheds are usually steeper and less pervious. Consequently, while greater total runoff may usually be expected, dry-season runoff is likely to be much lower because there is less opportunity for ground storage.

23. Temperature.—The effect of temperature on the amount and distri-

[1] See Reference 13 of Sec. 25.

bution of runoff is felt principally through its influence on evaporation, as previously explained. There are, however, several ways in which freezing temperatures can have an important bearing on the runoff.

In northern climates, the absence of the usual fall snows will allow the ground to become deeply frozen and, if ground water is close to the surface, it may become solidified and unavailable for winter runoff. If the ground freezes below the plane of saturation, very little winter precipitation can be absorbed by the soil, the precipitation remaining on the surface in the form of ice and snow or passing off as surface flow. Between periods of rainfall, in such cases, the flow of streams is produced entirely from ground water which decreases in rate of flow until the spring thaw opens the surface to seepage flow. Frozen ground is not usually impervious, unless covered with ice or unless freezing has reached ground water. A heavy coating of early snow will protect the ground from deep freezing and, if the depth of freezing has not reached groundwater, winter thaws will be absorbed by the soil.

When ice first forms on the surface of streams, it suddenly increases friction, and some of the flowing water is used to build up slope and area of waterway to maintain the required discharge capacity. The result is a sharp drop in discharge, which continues until the winter regimen of the stream is established.

In extreme northern climates, lakes and streams of little slope freeze to a considerable thickness, and the amount of water thus held in storage by streams and shallow lakes forms a large part of the winter flow. W. G. Hoyt [2] cites an example showing the possibility of thus holding back the equivalent of more than 20 per cent of the winter flow over a two-months' period following the freeze-up. Ice forming on deep lakes has no material effect on stream flow if the capacity of the outlet is not thereby reduced. The freezing of very shallow streams in wide valleys may extend below the bed of the stream, in which case the ground flow is effectually cut off and the discharge reduced to surface runoff.

24. Seepage and other Losses.—Serious loss of water occurs frequently from dams on porous foundations, particularly in the arid regions, and from very long, unlined canals.

In the absence of rock formations, seepage from reservoirs depends upon the elevation of the water table and the character of the underlying geological formation. The water table in the arid regions is at considerable depth and as much as 30 per cent of the flow has been lost in cases where a dam has been constructed on particularly pervious foundations. Excessive seepage, in some cases, has required the lining of the bottoms of small reservoirs.

A. P. Davis [3] gives the following table showing seepage losses from the Deer Flat Reservoir. It will be noted that the seepage steadily decreased with the length of service. The improvement is attributed to the filling of the subsoil with water, resulting in a rise of the water table, and not to any actual tightening of the soil.

[2] U. S. Geological Survey, Water Supply Paper No. 337, 1913.
[3] Eng. News-Record, Vol. 80, p. 663, 1918.

TABLE V

SEEPAGE LOSSES IN DEER FLAT RESERVOIR, IDAHO

Year	Mean Acreage Submerged	Maximum Acreage Submerged	Evorapation in Acre-feet	Total Loss in Acre-feet	Seepage Loss in Acre-feet	Average Seepage Loss	
						Acre-feet per Day	Inches per Day
1909	1,355	2,500	4,750	57,600	52,850	39.0	1.28
1910	3,002	3,900	10,500	95,483	84,983	28.3	0.93
1911	4,459	6,300	15,600	150,838	135,238	30.3	1.00
1912	4,625	7,000	16,200	85,089	68,889	14.9	0.49
1913	5,250	8,200	18,400	89,489	71,089	13.5	0.44
1914	5,337	8,400	18,700	82,084	63,384	12.0	0.40
1915	5,123	8,100	17,900	67,400	49,500	9.7	0.32
1916	4,820	6,900	16,900	43,970	27,070	5.6	0.18
1917	4,500	7,550	11,000	32,400	21,400	4.8	0.16
Mean.	4,274	6,540	14,440	78,150	63,820		
Total..	38,471	58,850	129,950	704,353	574,403		

Certain rock formations make poor foundations for high dams. Davis states that gypsum deposits and limestone deposits showing evidences of caves are productive of excessive leakage and that volcanic rock and coarse-grained sandstone should be regarded with suspicion.

In arid regions the losses from unlined canals and reservoirs may be excessive when the underlying materials are pervious. From a study of experiments by E. A. Moritz [4] and others, Etcheverry,[5] gives the following average conveyance losses [6] in unlined canals, of ordinary ratios of width to depth, the larger values being for canals less than five years old.

TABLE VI

CONVEYANCE LOSSES IN CUBIC FEET PER SQUARE FOOT OF WETTED PERIMETER FOR CANALS NOT AFFECTED BY THE RISE OF GROUND WATER

Character of Material	Cubic Feet per Square Foot in 24 Hours
Impervious clay loam	0.25 to 0.35
Medium clay loam underlaid with hardpan at depth of not over 2 to 3 ft. below bed	0.35 to 0.50
Ordinary clay loam, silt soil, or lava ash loam	0.50 to 0.75
Gravelly clay loam or sandy clay loam, cemented gravel, sand and clay	0.75 to 1.00
Sandy loam	1.00 to 1.50
Loose sandy soils	1.50 to 1.75
Gravelly sandy soils	2.00 to 2.50
Porous gravelly soils	2.50 to 3.00
Very gravelly soils	3.00 to 6.00

Most of the experiments upon which Table VI is based were made in canals in the arid regions, where the ground water is very low and is present

[4] Eng. Record, Vol. 70, p. 402, 1913.
[5] Irrigation Practice and Engineering, Vol. II, McGraw-Hill Book Co., 1915.
[6] Including about 5 to 10 per cent of evaporation losses.

during the irrigation season only. Canals for water power operate continuously, and the wet-season seepage losses would ordinarily be considerably less, owing to high ground water.

The foregoing applies only to conditions existing in the arid regions of the western United States where the ground-water level is frequently at a very great depth below ground surface. Even there, any particular case may depart widely from the figures given in Table VI. If the position of ground water, the topography of the surface, and the effective size and porosity of the material are determined, it is entirely possible to compute, for any particular case, with a fair degree of accuracy, the amount of seepage which may be expected. The methods to be used for such determinations of seepage are described in detail in "The Design of Earth Dams," by Joel D. Justin, Trans. Am. Soc. C. E., Vol. LXXXVI.

25. Bibliography.—(See also Sections 7 and 16.)

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6. The Effects of Ice on Stream Flow, by W. G. Hoyt. U. S. Water Supply Paper No. 337, 1913.
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CHAPTER IV

ESTIMATING STREAM FLOW

By William P. Creager

26. General.—All estimates of stream flow are based upon stream-gaging records, obtained at the site, near the site, or on neighboring streams. It is not possible to estimate stream flow, solely upon rainfall records, with a degree of accuracy sufficient for practical purposes.

The main purposes for which estimates of usable stream flow are made may be stated as follows:

(1) To estimate the average annual energy output of the development;
(2) To estimate the additional energy provided by a proposed storage reservoir;
(3) To estimate the minimum annual energy output;
(4) To determine the capacity of a storage reservoir to equalize the flow to a given minimum;
(5) To estimate the minimum daily output without storage.

For an equal degree of accuracy, the amount of data required for these purposes is least for the first and increases in the order in which they are listed. In Table I is given the probable deviation of short-term precipitation records from the true mean. There is reason to believe that stream-flow records are subject to no greater errors. This applies to the probable errors that are involved when runoff records are used to determine average annual energy output and the average annual energy provided by storage reservoirs, Items 1 and 2.

In the case of estimates of flow governing minimum annual output and minimum daily output, and estimates intended to determine the capacity of reservoirs to equalize the flow to a given minimum, the problem is one of probabilities, the general solution of which is given in Sec. 86. The minimum yearly and daily output can be defined only in terms of probabilities, the occurrence of which may be expected not oftener than once in a given period of years. The determined size of the reservoir, to equalize the flow to a given minimum, is that size which will be deficient not oftener than once in a given period of years.

The methods commonly used to compute the amount and distribution of stream flow which may be used to develop power vary with the extent and character of available data. Such data may be grouped as follows:

(1) Long-term stream gagings at the site.
(2) Short-term stream gagings at the site.
(3) No stream gagings at the site.

All stream-flow records should be corrected for possible diversion from the watershed for irrigation, water supply, and other purposes, and for increased evaporation losses if large storage reservoirs are to be used.

27. Stream Gagings at the Site.[1]—While stream-gaging stations have been established at thousands of places in the United States, records of stream flow at single stations, covering a long term of years, are comparatively few.

The most important data on stream flow are published in the Water Supply Papers of the U. S. Geological Survey, Washington, D. C. The Corps of Engineers, U. S. Army, and many of the States also publish yearly tabulations of stream gagings and general studies of various streams. A few reliable records may also be obtained from water power and water supply companies and city water supply commissions and engineers.

The United States Weather Bureau publishes daily river stages of many streams, without data as to discharge. Studies of a number of large streams may be found in The Tenth Census of the United States, Vols. XVI and XVII.

The results of stream-flow measurements are given in the U. S. Geological Survey Water Supply Papers which are published annually in 12 parts, each part covering an area whose boundaries coincide with natural drainage features as indicated below:

Part
I. North Atlantic Coast basins;
II. South Atlantic Coast and eastern Gulf of Mexico basins;
III. Ohio River basin:
IV. St. Lawrence River basin;
V. Upper Mississippi River and Hudson Bay basins;
VI. Missouri River basin;
VII. Lower Mississippi River basin;
VIII. Western Gulf of Mexico basin;
IX. Colorado River basin;
X. Great Basin.
XI. Pacific Coast basins in California;
XII. North Pacific Coast basins.

Other Water Supply Papers, published periodically, contain data on water resources of special districts, including studies of hydrology, hydrography, flood flows, analyses of river waters, water powers and water power possibilities, underground flow, river profiles, and other data of interest.

Water Supply Paper No. 340 contains a list of all Water Supply Papers and other publications of the U. S. Geological Survey that are applicable to the various districts listed above, up to and including 1913. A list is given of all rivers that had been gaged previous to that date, and of the years covered by the individual records. On page 4 of this paper is given a

[1] See Chapter XXXIV for description of steam-flow gagings.

handy reference to the serial number of Water Supply Papers containing stream measurements for each district by years.

A list of rivers that have been gaged in each district, and of the years covered by the individual records, may also be found at the end of each of the later Water Supply Papers applicable to the district in question.

A publication of the Survey, entitled "A List of the Publications of the United States Geological Survey," dated August, 1921, contains a list of all Water Supply Papers published to that date.

The probable accuracy of stream gagings is usually stated in the reports. A factor of safety should always be applied to results obtained from such records, to cover all possible inaccuracies of measurements and, as explained heretofore, to compensate for the possibility that the years of record do not correspond to average conditions or include years of reasonably low flows.

28. Extensions of Stream Gagings at the Site.—The period of stream gagings may be extended to include additional years, by establishing a relation between the flow indicated by such gagings and the simultaneous flow of a neighboring stream on which records cover a longer period. In Fig. 21 is given a runoff-relation curve for the Little Swatara and Brandywine Creeks, Pa., for three years of simultaneous records. Each dot indicates a concurrent average monthly discharge per square mile of drainage area. After the whole period of overlapping records has been plotted, an average curve is drawn through the points. This curve is used to alter the longer records of the neighboring stream to indicate the probable flows in the stream under consideration for years for which records are not available. The better low-season runoff of the Brandywine and the better wet-season runoff of the Little Swatara are clearly defined in Fig. 21.

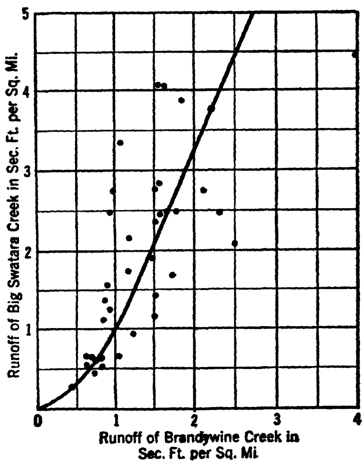

FIG. 21.—A Typical Runoff Relation Curve.

Suppose that on a stream, A, under consideration, there are ten years of records and, on a neighboring stream, B, there are twenty years of records, four years of which overlap those of stream A. If a runoff relation is established between the two streams, as previously described, and this relation is used to extend the records of stream A, the total period of estimated stream flow for A would be $10 + 20 - 4 = 26$ years. The probable error involved in the use of only the ten years of record of stream A, if the estimate is used for average output, is indicated by Table I to be about 7 per cent. If a perfect runoff relation were established between the two streams, the resulting twenty-six years of records would have a probable error of only about 2.4 per cent.

The following equation gives the maximum permissible error in the estimated runoff relation between the two streams to result in a benefit from the extension of the records of stream A.

$$x_3 = \frac{100y_2}{y_2 - y_1}\left(1 - \frac{100 - x_1}{100 - x_2}\right),$$

Where x_3 = the permissible error in the runoff-relation curve,
x_1 = the probable per cent error in the records of stream A if used alone,
x_2 = the probable per cent error in the extended records of stream A, if a perfect runoff-relation curve were used,
y_1 = the years of record of stream A,
y_2 = the years of record of stream B.

Substituting in this equation, we have:

$$x_3 = \frac{100 \times 20}{20 - 10}\left(1 - \frac{100 - 7}{100 - 2.4}\right) = 9.43 \text{ per cent.}$$

If it is thought that, in establishing the runoff relation, there may be an accumulative error of 9.43 per cent or greater, no benefit will be derived from the extension and the ten years of stream A should be used alone. The probable error in the runoff-relation curve can be based only on the judgment of the engineer and is affected considerably by the length of simultaneous gagings and the relative runoff characteristics of the two streams. The smaller the average distance of the dots from the curve, the greater the degree of accuracy. The curve of Fig. 21 is fairly well defined but a probable error of 10 or 15 per cent would not seem unreasonable. The curve of Fig. 22 is poorly defined and might be in error as much as 20 or 30 per cent.

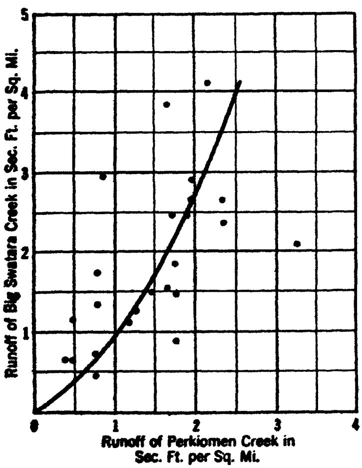

FIG. 22.—A Poorly Defined Runoff Relation Curve.

If the runoff characteristics of the two streams are materially different, a more exact runoff-relation curve may be established by plotting a runoff-relation curve for each season or for each month. In such cases, weekly averages may be used instead of monthly averages, in order to obtain a greater number of points from which to estimate the curve.

A number of methods have been proposed [2] for estimating the amount and

[2] See Bibliography, Sec. 31.

distribution of stream flow from an established relation between rainfall and runoff and applying this relation to long-term rainfall records. Such methods cannot be of any value unless the relation between rainfall and runoff is based on at least several years of simultaneous runoff and rainfall records. Most of the methods are rational but are not intended to be exact. Because of the numerous factors affecting the relation between rainfall and runoff, which have not yet been coordinated to an extent sufficient for exact analysis, it is probable that the errors involved in extending runoff data by this means may be very great, and the chief objection to the use of such methods consists of the difficulty in the determination of the probable degree of accuracy. Moreover, those methods that include a consideration of all major agencies affecting runoff involve an amount of labor and time which ordinarily cannot be devoted to such problems.

In Table VII is given a relation between the rainfall and runoff of the Miami watershed above Dayton, Ohio.[3] Rainfall and runoff years [4] were used in establishing the relation. Cols. 2 and 4 are reproduced in Fig. 23.

TABLE VII

ANNUAL RAINFALL, RUNOFF, AND TEMPERATURE, MIAMI RIVER ABOVE DAYTON, OHIO

1 Year Ending Sept. 30	2 Inches Rainfall	3 Degree F. Temperature	4 Actual Inches Runoff	5 Inches Runoff Computed from Equation of Sec. 29
1894	30.7	54.7	4.9	10
1895	34.0	53.0	3.7	12
1896	46.2	54.0	8.1	16
1897	33.3	53.4	12.8	8
1898	44.3	55.0	14.7	15
1899	34.2	53.2	9.7	9
1900	35.1	54.4	6.6	9
1901	30.1	53.4	5.6	7
1902	31.6	51.1	5.8	10
1903	37.1	53.6	12.6	11
1904	39.1	49.8	13.1	15
1905	38.5	51.5	7.1	13
1906	33.2	52.9	9.2	9
1907	43.1	51.9	17.2	16
1908	37.7	53.1	17.7	11
1909	39.3	53.2	13.1	12
1910	36.3	52.3	15.1	12
1911	39.8	53.7	13.9	12
1912	43.8	50.8	23.1	18
1913	42.9	54.0	24.4	14
1914	32.3	53.3	8.3	8
1915	41.8	52.1	12.1	15
1916	39.9	53.2	19.2	13
1917	35.7	51.1	11.4	12
1918	36.8	50.3	9.4	14
Mean	38.07	52.76	11.87	12

[3] Compiled from "Rainfall and Runoff in the Miami Valley," by I. E. Houk. Tech. Report, Miami Conservancy District, Part VIII, 1921.

[4] See Sec. 4.

Figure 23 is typical and indicates that the variation in the amount of annual runoff is affected more by the distribution of the rainfall during the year than by the amount of rainfall. The average annual temperature, indicated in Col. 3, being so nearly constant, probably had little influence in the variation of runoff. It is evident that such a relation, if used for extending the period of runoff records, may lead to considerable error for individual years although the general average may approximate the truth.

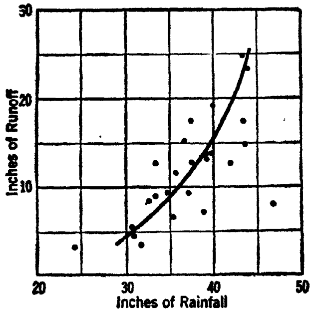

FIG. 23.—Relation between Annual Rainfall and Runoff, Miami River, above Dayton, Ohio.

29. No Stream Gagings at the Site.—If no stream gagings have been made at the site of the proposed development, it is necessary to base estimates of steam flow on other gagings in the vicinity. If gagings have been made on the stream a short distance from the site, the problem presents no features of great difficulty. In such cases it is considered that, if the two drainage areas have no appreciable differences in rainfall and runoff characteristics, the runoff per square mile will be the same for each.

The area of the watershed may be obtained from various sources. The topographical maps of the U. S. Geological Survey usually supply the best information obtainable. A number of other maps have been published by the Survey. The U. S. Post Route Maps and various other State and County maps are available for this purpose. Surveys are sometimes necessary if accurate maps are not available.

If the only gagings available are those made at a considerable distance from the site, and particularly if they are on another stream, estimates of stream flow at the site must be based on such gagings, modified to compensate for differences in the areas and the rainfall and runoff characteristics of the two watersheds. A practical solution of this case calls for the best judgment of the most experienced engineers, backed by a careful examination of the drainage areas, a study of all published data relating to their physical and climatological features, and an intimate knowledge of the effect of the many controlling factors mentioned in the foregoing sections.

The errors that may result from the assumption of the same runoff per square mile from two drainage areas, without modifications for differences in the characteristics of the two watersheds, are well indicated by the comparison between the flow per square mile from the Big Swatara Creek and Brandywine Creek watersheds, shown in Fig. 21. Errors as great as 100 per cent may be expected in some cases. Figure 21 clearly indicates the relative imperviousness of Big Swatara Creek, resulting in greater total runoff but less opportunity for ground storage, and hence small low-season flows.

If gagings have been made on the stream both above and below the site of the proposed development, estimates of stream flow at the site can often be made with considerable accuracy by interpolation.

Short-term gagings on the stream relatively near the site should always be used in preference to longer-term gagings on a different stream, because, for the former, a part of the watersheds are identical. The short-term gagings on the same stream may be extended, however, by means of a runoff-relation curve as explained in Sec. 28.

If there are no gagings at the site or in the vicinity, reliable estimates of stream flow cannot be made. While admitting the possibility of extending runoff records with some degree of accuracy by the establishment of a relation between rainfall and runoff by methods referred to in Sec. 28, engineers generally agree that it is impossible to establish such a relation without the aid

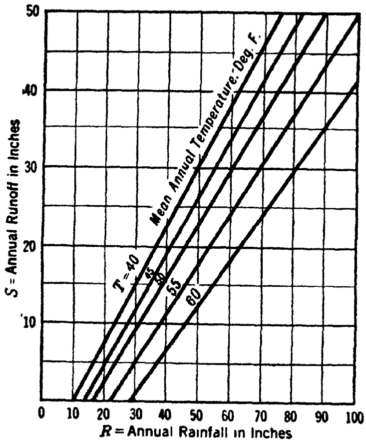

FIG. 24.—Vermeule's Approximate Relation Between Annual Rainfall, Temperature and Runoff.

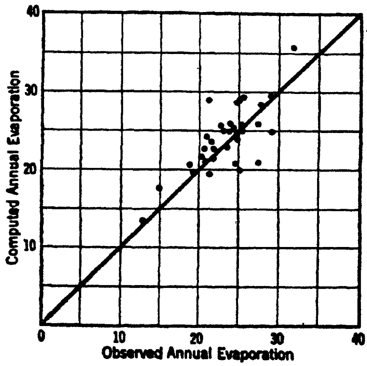

FIG. 25.—Comparison of Observed Evaporation and Evaporation Computed by Vermeule's Equation.

of at least several years of runoff records, particularly for cases where it is necessary to know the distribution of runoff.

Vermeule's equation [5] for annual runoff is

$$S = R - (11 + 0.29R)M.$$

Where S = total annual runoff in inches;
R = total annual rainfall in inches;
M = a factor which depends upon the mean annual temperature.

Vermeule's recommended value of M has been used in plotting Fig. 24. This equation is the result of studies to determine the effect of forests on evaporation and, while extreme accuracy is not claimed for it, the equation has received wide publicity.

Figure 25, showing a comparison between observed and estimated values

[5] See Reference 9, Sec. 25.

of mean annual evaporation ($R - S$), computed from Vermeule's equation, has been plotted from his tabulations of rivers in this country and abroad. An error of 50 per cent ordinarily would be possible in the application of the equation to determine the average annual runoff for a considerable period of years. In the arid region, greater errors might be made.

The runoff of the Miami River above Dayton, Ohio, computed by Vermeule's relation, is given in Col. 5 of Table VII. The observed and computed runoff, shown by Cols. 4 and 5, have been compared in Fig. 26. It is seen that Vermeule's equation may be several hundred per cent in error if used for single years, although a close agreement may exist between the observed and computed mean for the period.

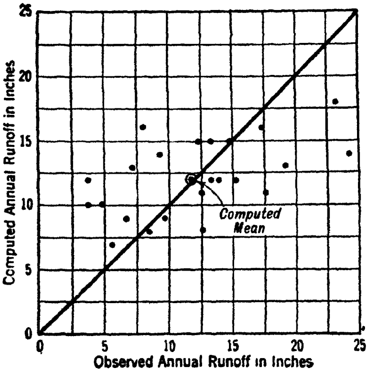

FIG. 26.—Relation between Observed Runoff of the Miami River above Dayton, Ohio, and Runoff Computed from Vermeule's Equation.

30. Storage and Pondage.—The demands for power ordinarily correspond in no degree to the varying natural flow of the stream. Therefore, unless a large part of the natural flow is to be wasted, storage and pondage must be provided to regulate the flow so that it may be made available at times when the generation of power is required.

Storage consists in the impounding of considerable of the excess runoff during seasons of surplus flow, for use during dry seasons. Ordinarily, the content of storage reservoirs are fed out in such a manner that, when they are added to the natural runoff from the intermediate area between the reservoir and the site of the development, the resulting flow at the development will correspond as closely as possible to the flow demanded for power.

When the reservoir is some distance from the development, it is impossible to regulate the outflow with sufficient accuracy to provide for sudden changes in load demand or to compensate for the varying runoff from the intermediate area. Therefore, a regulating body of water, or pondage, is needed directly at the plant. This may be drawn upon quickly to suit sudden changes in load demand and to compensate for inaccuracies in operation of the storage reservoir. When storage reservoirs are not provided, pondage is necessary to regulate the natural flow to suit the variation in daily or weekly load demand.

Usually, the hourly demand is quite variable, and the average demand during a work day is often materially different from that of Saturdays and Sundays. The average weekly demand, however, is usually fairly constant. The duty required of pondage, without storage, is therefore usually that of regulating the weekly flow to suit the variation in load demand from the average weekly demand.

In general, the word "storage," is used to indicate the building up of the

low natural flow of the stream to a uniform discharge, and the word " pondage " is used to indicate the regulation of the resulting uniform flow, or the natural flow if there is no storage, to suit variations in weekly load demand. If storage is at the site of the development, it will also provide the necessary pondage.

31. Bibliography.—References 1, 7, 8, 9, 10, and 11 of Sec. 25 contain special citations of computations of runoff from rainfall.

CHAPTER V

FLOOD FLOWS

By William P. Creager

32. General.—As the magnitude of a flood is affected by an infinite variety of conditions, the chances of the maximum possible flood from a given drainage area occurring within the life of a development is infinitely remote. According to the law of probabilities, the greater the flood the less the frequency of its occurrence. As the maximum flood of which a drainage area is capable of producing is beyond the realm of practical consideration, it is customary to define floods in terms of probable frequency. Thus, a flood that probably will be equaled or exceeded, on the average, only once in a period of n years is called an "n-year flood."

As the cost of a dam usually increases with the magnitude of the flood which it is designed to accommodate, it is possible, where no loss of life would be involved in failure, to design for a 50-year flood or even for a 20- or 10-year flood, if it can be shown that it would be more economical to repair the flood damage at intervals than to incur the additional expense necessary to accommodate a larger flood. Such decisions are always influenced, however, by the fact that an n-year flood is an n-year probability only, and that a larger flood might occur at any time or even be repeated at short intervals. A 1000-year flood, or even a 10,000-year flood, is frequently used for the design of important structures above large communities.

For the same development, different floods may be used for the design of various structures, according to their importance and their margin of safety. An earthen embankment, which will not stand overtopping without immediate failure, should be designed on the basis of a larger flood than a concrete dam, which has a much greater margin of safety. The purchase of rural lands for flowage purposes, to an elevation corresponding to the flood for which the dam is designed, would ordinarily be a foolish investment, particularly when the damage to such lands once in a great number of years is small compared with the compound interest on the money necessary to purchase them.

The determination of the flood characteristics of a stream involves:

(*a*) A study of the frequency and magnitude of past floods if long-term gagings on the stream are available;

(*b*) Consideration of physical indications of the magnitude of past floods on the stream beyond the period of gagings;

(*c*) An examination of the flood-producing characteristics of the water-

shed compared with those of other watersheds on which considerable data are available.

33. Frequency Studies.—The frequency of floods during a long period of gagings of a stream is often the best indication of probable future floods, and a study can be made on a mathematical basis by the law of probabilities, as indicated in Sec. 86. This law may be applied to the study of floods by either of the following methods:

(*a*) *Basic-stage Method.*—A consideration of all floods that exceeded a given basic stage during the period of records.

(*b*) *Yearly-flood Method.*—The use of only the maximum flood during each year of record.

For the basic-stage method it is necessary to define what constitutes a flood. A single flood, for use in frequency studies, may be defined as an increase in flow above an assumed fixed basic stage, irrespective of the number of days the flow remained above that stage or the number of peaks and valleys in the flow before the discharge again receded below the basic stage.

The choice of basic stage influences the number of floods to be considered, it being obvious that an extremely high or an extremely low basic stage would result in the consideration of very few floods. Theoretically, the best basic stage is that which results in the greatest number of floods, since by such means we have the greatest amount of data and hence the most accurate result. Unfortunately, however, in many rivers, the basic stage resulting in the greatest number of floods corresponds to a flow so low as to be entirely below the flood class, and some other method of fixing the proper basic stage must be devised.

It is generally conceded that lake and swamp areas have a much greater influence on the frequency and magnitude of small floods than on those of large ones. Consequently, it is becoming more generally recognized that in all flood-probability studies a greater weight should be attached to the larger floods. It is therefore advisable to eliminate from consideration as many of the smaller floods as possible without reducing too greatly the number of floods considered. In general, a basic stage equal to or slightly lower than the lowest maximum yearly flood is recommended.

Curve *A* of Fig. 27 shows the probable frequency of floods on the Tennessee River at Chattanooga, Tenn. The calculations for this curve are used as an example in Sec. 86 to explain the law of probabilities. Curve *A* is reproduced from Fig. 111. The calculations for Curve *A* were based on the past frequency of floods above a basic stage of 100,000 sec.-ft. or 4.67 sec.-ft. per square mile.

Curve *B* of Fig. 27 is based on the yearly-flood method, the calculations for which follow the method given in Sec. 86 for the basic stage. The calculations for yearly floods are not given here, but, when made, should be recorded in Col. 2 of Table XXVI, which will then total an amount equal to the number of years of records and m will equal y in all the equations. In the yearly-flood method, many floods that should be considered in a true probability study are excluded. The method involves considerably less work than the true, or basic-stage method, but should only be used for approximate calculations,

Curves *A* and *B* draw together and are coincident for the maximum recorded flood, but the projected curves draw apart, the yearly-flood curve giving results which are considered too low. The yearly-flood method is quite inaccurate for studies of the frequency of the lesser floods, as indicated in Fig. 27.

Great accuracy in frequency curves can be obtained only where long-term gagings are available. In all cases, such information as may be gleaned from frequency studies should be backed up by whatever other investigations

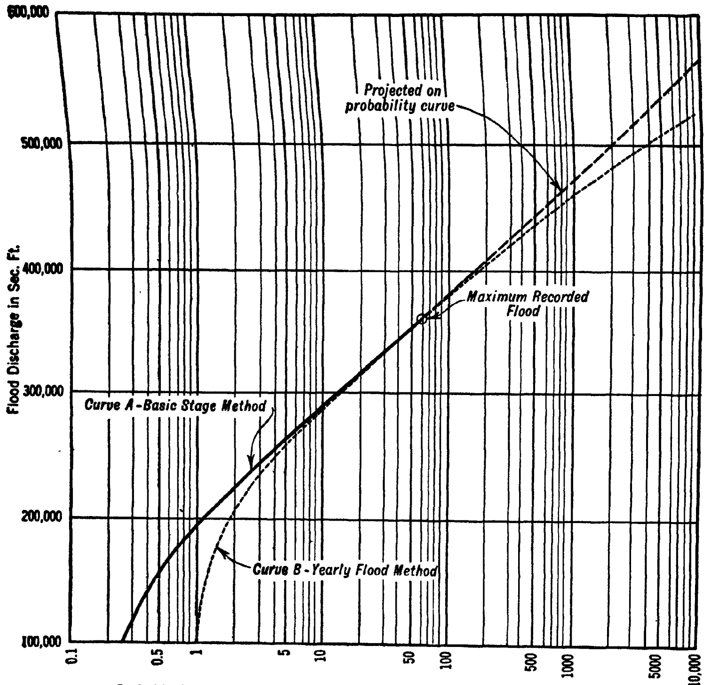

FIG. 27.—Frequency Curve, Tennessee River, at Chattanooga, Tenn.

available data may permit, and an ample factor of safety should always be provided, particularly if loss of life or heavy financial loss would be involved in the destruction of a structure by an excessive flood. It should be remembered that such studies are probabilities only and that floods of great magnitude may be imminent.

34. Physical Indications of Past Floods.—Authentic Federal and State Government records of high water, extending over long periods, may be obtained for many streams.[1] Such records are also often available from mill operators and officials of municipalities. In the great majority of cases, how-

[1] U. S. Geological Survey, U. S. Weather Bureau, Engineers' Corps, U. S. Army, etc.

ever, the elevation of record high water must be determined from the observations and traditions of residents and from physical indications on the banks of the stream.

Observations and traditions of residents should be regarded with caution. Individual reports of untrained observers are subject to great error and strange to say, are often of doubtful veracity, as the desire to report high water a little higher than that reported by a neighbor is often, among certain classes, greater than the love of truth. Unfortunately, also, reports are sometimes biased by a desire to give an impression of great or small floods, whichever, in the opinion of the observer, will better serve his interests. However, credence may be given when a number of observations closely agree and are referred to definite objects, such as sills of doors and windows or nails driven for reference.

Confirmation may be obtained from the elevation of depositions of brush, logs, or alluvial matter, scars from floating logs on banks and large trees, and whatever other indications of high water may be discovered. High water, in an alluvial valley which has been formed from the sediment deposited from floods, is, of course, always higher than the surface of such deposits.

The elevation of record high water having been fixed, there are four methods by which an estimate of the corresponding discharge can be made:

(1) The head on a dam which existed at the time of such high water can be determined and from this the discharge over that structure can be computed by one of the well-known weir formulae;[2]

(2) In unusual cases where the loss of head at contracted openings between bridge abutments has been observed, the approximate discharge can be computed from the expected loss at such openings.[3]

(3) If a considerable stretch of straight river having a nearly uniform cross-section and slope is available, a fairly close estimate of the discharge can be made by use of Kutter's formula for the flow in open channels, particularly if accurate current-meter measurements of smaller floods have been made in order to determine the coefficient of roughness of the channel.[4]

(4) By the projection of a rating curve to the elevation of high water.[5] This method, however, is only available as a rough indication of the corresponding flood, unless the cross-section of the river is particularly regular and the discharge measurements used in plotting the curve cover a range including floods amounting to considerable proportions.

35. Flood-flow Equations.—In Table VIII is given a list of unusual flood discharges on United States rivers, gathered from a number of sources. It is thought that the list includes practically all of the remarkable floods mentioned in published data. As many of the data indicated average twenty-four-hour discharge, the momentary maximum flow in such cases was estimated by use of Fuller's equation, Eq. (2) of this section. These floods have been plotted in Fig. 28.

[2] See U. S. Geol. Survey Paper 200, by R. E. Horton.

[3] See Report of the Chief Engineer, Miami Conservancy District, Vol. 1, March, 1916, p. 63.

[4] See Sec. 73.

[5] See Chapter XXXIV.

TABLE VIII

UNUSUAL FLOOD DISCHARGES, UNITED STATES RIVERS

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
1	Black Warrior	Tuscaloosa	Alabama	4,900	141,000	28.8
2	Tennessee	Florence	"	30,800	500,000	16.2
3						
4						
5						
6	Salt	....	Arizona	12,000	296,000	24.7
7	Salt	McDowell	"	6,200	159,000	25.7
8	Verde	McDowell	"	6,000	144,000	24.0
9	Salt	Roosevelt Dam	"	5,750	207,000	36.0
10	Canyon Diablo	Leupp	"	544	44,600	82.0
11	Troxton Canyon	E. of Kingsman	"	450	49,500	110.0
12	Canyon Diablo	Arch Bridge	"	340	35,300	104.0
13	Care Creek	Phoenix	"	200	25,000	125.0
14	Pinal	Globe	"	30	13,200	440
15	Chase Creek	Of Gila River	"	20	12,900	647
16						
17						
18						
19	White	Clarendon	Arkansas	19,000	320,000	16.84
20						
21						
22	Sacramento	....	California	22,500	576,000	25.6
23	Sacramento	Red Bluff	"	10,400	254,000	24.4
24	Sacramento	Iron Canyon	"	9,290	218,000	23.5
25	Feather	Oroville	"	3,640	187,000	51.3
26	Eel	Scotia	"	3,071	290,000	94.4
27	N. Feather	Big Bend	"	1,940	109,300	56.3
28	American	Fair Oaks	"	1,910	105,000	55.0
29	American	Folsom (near)	"	1,900	189,000	99.5
30	Kings	Sanger	"	1,694	59,800	35.3
31	San Joaquin	Hamptonville	"	1,640	73,000	44.5
32	Tuolumne	Lagrange	"	1,500	52,600	35.1
33	Yuba	Smartsville	"	1,200	111,000	90.9
34	Stanislaws	Knights Ferry	"	930	57,200	61.2
35	McCloud	Baird	"	669	55,000	82.2
36	Santa Ynez	Lompoc (near)	"	750	41,500	55.3
37	San Jacinto	Elismore	"	717	140,000	195
38	Putah Creek	Winters	"	655	60,000	91.6
39	McCloud	N. Gregory	"	608	41,500	68.3
40	Stony	N. Fruto	"	601	29,300	48.8
41	San Lois Rey	Oceanside	"	565	95,400	169
42	Los Angeles (Tributaries)	Los Angeles	"	534	38,000	71.2
43	San Diego	San Diego	"	434	75,100	173
44	Calavares	Jenny Lind	"	395	69,500	176.2
45	San Diego	Santee	"	375	70,000	187
46	San Dieguito	Bernardo	"	299	70,000	241
47	Mattole	New Petrolia	"	249	55,500	223
48	Bear	Van Trent	"	263	25,800	98.1
49	Salinas (Tributaries)	Soledad	"	238	19,000	80.0
50	Smith	Crescent City	"	227	42,500	187
51	San Gabriel	Azusa	"	222	40,000	180
52	Sespe Creek	Sespe	"	216	18,600	86

TABLE VIII—*Continued*

tem No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
53	Arroyo Seco	N. Soledad	California	215	13,300	61.9
54	San Luis Rey	Mesa Grande	"	209	58,500	280
55	San Diego	San Diego	"	207	14,900	72
56	Santa Ynez	N. Santa Barbara	"	207	94,500	457
57	Santa Ana (Tributaries)	Mentone	"	199	29,000	146
58	Coyote	Madrone	"	197	16,100	127
59	Santa Ana	Mentone	"	189	29,100	154
60	San Diego	Cap. Gr. Dam Site	"	189	36,300	192
61	Sweetwater	Sweetwater Dam	"	181	45,500	251
62	Cottonwood	Moreno Dam	"	120	15,400	128
63	Sweetwater	Dehesa (near)	"	112	24,300	217
64	Santa Ysabel	Ramona	"	110	28,400	258
65	San Jacinto	San Jacinto	"	108	30,000	278
66	Churn Creek (Group)	....	"	100	10,000	100
67	Otay	Lower Otay Dam	"	99	23,600	238
68	Putah	N. Guenoc	"	91	18,100	199
69	San Vincent Creek	Foster	"	74.9	18,600	248
70	Jamul Creek	Otay (near)	"	69.8	17,700	259
71	San Jacinto	Hemet Reservoir	"	65.8	9,550	145
72	Los Angeles	Los Angeles (near)	"	62	7,250	117
73	Lytle Creek	San Bernardino	"	60	16,000	267
74	Santa Maria Cr.	Ramona	"	57.3	7,150	125
75	Santa Ysabel	Mesa Grande	"	53	20,900	395
76	Sweetwater	Descano	"	43.7	14,300	326
77	Moosa Cr.	Bonsall (near)	"	31	8,300	269
78	Arroyo Seco	Pasadena (near)	"	30.5	11,400	374
79	Arroyo Seco	Pasadena (near)	"	16.4	3,150	192
80	Boulder Cr.	Julian (near)	"	12.0	2,600	217
81	Malabu Cr.	Cabasas	"	97.0	10,200	105
82	San Luis Rey	Pala	"	322	75,000	233
83						
84						
85						
86						
87						
88	Arkansas	Pueblo	Colorado	1,740	98,000	57
89	Two Buttes	Arkansas	"	900	35,100	39
90	Purgatory	Trinidad	"	742	45,300	61
91	St. Charles	Pueblo	"	482	72,000	149
92	Fountain	Pueblo	"	930	50,000	54
93	Cherry (All)	Denver	"	445	24,900	56
94	Cherry	Denver	"	200	20,000	100
95	Cherry	Denver	"	175	10,000	57
96	Dry	Pueblo	"	70	21,000	300
97	Eight Mile Cr.	Pueblo (near)	"	65	10,000	154
98	Rock Cr.	Pueblo (near)	"	59	54,000	913
99	Turkey Cr.	Pueblo (near)	"	48	9,000	48
100	Pecks Cr	Pueblo (near)	"	34.4	19,400	564
101	Boggs Cr.	Pueblo (near)	"	26	15,100	582
102	Coal Cr.	Pueblo (near)	"	22.3	3,720	167
103	Brush Hollow Cr.	Pueblo	"	21.9	4,300	243
104	Rush Cr.	Pueblo (near)	"	19.6	4,650	238
105	N. Arroyo	Pueblo (near)	"	15.6	6,400	410

TABLE VIII—*Continued*

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
106						
107						
108						
109	Connecticut	Hartford	Connecticut	10,270	205,000	20
110	Pequonnock	Bridgeport	"	25	3,920	157
111			"			
112			"			
113			"			
114			Delaware			
115			"			
116			Florida			
117			"			
118			"			
119	Savannah	Augusta	Georgia	7,500	300,000	40
120	Chattahoochee	West Point	"	3,300	88,800	26.9
121	Rhine	Macon	"	2,570	96,500	37.5
122	Oconee	Greenshore	"	1,100	68,200	62
123	Broad (of Ga.)	Carlton	"	762	60,000	78.7
124			"			
125			"			
126			"			
127			Idaho			
128			"			
129			"			
130	Ohio	Cairo	Illinois	233,000	1,400,000	60
131			"			
132			Indiana			
133			"			
134			"			
135	Devil's	N. Mouth	Iowa	143	186,000	1300
136	Dry Run	Decorah	"	22.3	16,050	720
137	Little Devil's	....	"	19	10,650	560
138	Panther	....	"	14	7,280	520
139			"			
140			"			
141			"			
142			Kansas			
143			"			
144			"			
145	Ohio	Paducah	Kentucky	205,700	1,440,000	7.0
146	Ohio	Louisville	"	90,600	770,000	8.5
147			"			
148			"			
149			"			
150	Kennebec	Waterville	Maine	4,270	151,000	35.4
151	Kennebec	Waterville (above)	"	2,700	131,000	48.6
152	Piscataquis	Foxcroft	"	286	22,200	77.6
153			"			
154			"			
155			"			
156	Potomac	Washington, D. C.	Maryland	11,500	470,000	40.9
157	Potomac	Point of Rocks	"	9,650	472,000	48.9
158	Gunpowder	....	"	302	25,000	82.8

TABLE VIII—*Continued*

tem No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
159	Rock Creek	near Washington, D.C.	Maryland	78	9,800	126
160	Lake Roland	Baltimore (near)	"	39	9,000	230
161			"			
162			"			
163			"			
164	Connecticut	Holyoke	Massachusetts	8,600	183,000	21.1
165	Westfield		"	356	53,000	149
166	Great	Westfield	"	350	72,000	206
167	Nashua		"	109	11,400	105
168	Fomer	Holyoke	"	13	2,840	218
169			"			
170			"			
171			"			
172			Michigan			
173			"			
174			"			
175			Minnesota			
176	Rock Creek	Ellisville	Mississippi	15	16,650	1110
177			"			
178	Rio des Perca	St. Louis (near)	Missouri	23.8	6,100	256
179	Rio des Perca	Clayton Rd., St. Louis	"	15.6	6,400	410
180			"			
181			"			
182			"			
183	Sun, North Fork	Augusta	Montana	600	32,400	54
184			"			
185			"			
186	Pennigewasset	Plymouth	New Hampshire	615	30,700	49.8
187			"			
188			"			
189			Nevada			
190			"			
191			"			
192			Nebraska			
193			"			
194			"			
195	Delaware	Stockton	New Jersey	6,850	255,000	37.1
196	Delaware	Riegelsville	"	6,430	177,000	29.5
197	Passaic	Dundee Dam	"	825	35,900	43.5
198	Raritan	Bound Brook	"	806	52,000	64.5
199	Pompton	Two Bridges	"	380	23,400	61.6
200	Ramapo	Mahwah	"	118	12,400	105.1
201	Wanaque	Pompton	"	101	8,450	83.6
202			"			
203			"			
204			"			
205	Verde	McDowell	New Mexico	6,000	166,000	27.6
206	Canadian	Taylor	"	2,832	91,000	32.1
207	Canadian	French	"	1,480	156,000	105
208	Mera	Weber	"	294	27,600	94
209	Mora	La Cuera	"	210	22,300	106
210	Turquillo	Mora Valley	"	160	16,000	100
211	Mora	Mora (below)	"	159	22,300	140

TABLE VIII—*Continued*

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
212	Gallins	Las Vegas	New Mexico	90	11,600	129
213	Cameron Cr	Hurley	"	44	5,500	125
214						
215						
216						
217						
218						
219	Hudson	Mechanicville	New York	4,500	120,000	26.6
220	Mohawk	Cohoes	"	3,472	99,000	28.5
221	Delaware	Port Jervis	"	3,250	127,000	39.0
222	Chemung	Elmira	"	2,050	138,000	67.1
223	Genesee	Mt. Morris	"	1,070	41,900	39.2
224	Schoharie	Fort Hunter	"	900	49,600	55.1
225	Delaware (E. Br.)	Hancock	"	920	72,000	78.3
226	Black	Lyons Falls	"	897	41,300	46
227	Delaware (W. Br.)	Hancock	"	680	43,000	63.3
230	Cattaraugus	....	"	467	25,000	53.6
231	Buffalo Cr	....	"	420	23,000	54.8
232	Esopus	Saugerties	"	417	55,000	132
233	Esopus	Mt. Marion	"	378	24,700	65.3
234	W. Canada Cr	Trenton Falls	"	376	36,300	96.5
235	W. Canada Cr	Hinckley	"	372	39,000	104.6
236	W. Canada Cr	Twin Rock Br	"	364	46,000	126
237	Croton	Croton Dam	"	339	30,000	88.5
238	Esopus	Kingston	"	324	20,400	63
239	Catskill	Woodstock	"	210	21,000	100
240	Fish (E. Br.)	Point Rock	"	104	8,400	80.5
241	Nine Mile	Stillville	"	63	7,820	125
242	Six Mile	Ithaca	"	46	8,500	185
243	Sawkill	Kingston	"	35	8,020	229
244	Trout Br	Brooksport	"	25	3,950	158
245			"			
246			"			
247			"			
248			"			
249			"			
250	Roanoke	Old Gaston	North Carolina	8,350	283,000	32.9
251	Yadkin	Salisbury	"	3,400	130,000	38.2
252	Catawba	Catawba	"	1,535	95,000	61.9
253	Catawba	Morganton	"	758	41,000	54.1
254	Little Tennessee	Jackson	"	675	57,500	85.3
255	Tuckaseegee	Bryston	"	662	38,700	58.5
256	Hiwassee	Murphy	"	410	22,200	54
257	Catawba	Bridgewater	"	370	55,500	150
258	Valley	Tomotia	"	106	10,300	97
259	Cane Cr	Bakersville	"	22	29,500	1341
260			"			
261			"			
262			"			
263			"			
264			"			
265			"			
266			North Dakota			

TABLE VIII—*Continued*

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
267			North Dakota			
268			South Dakota			
269			"			
270	Muskingum	Marietta	Ohio	78,500	2,510,000	32
271	Ohio	Cincinnati	"	75,800	660,000	8.7
272	Miami	Miami	"	3,940	384,000	97.4
273	Miami	Miamisburg	"	2,722	257,000	94.5
274	Miami	Dayton	"	2,450	246,000	100.4
275	Scioto, Lower	Columbus	"	1,570	146,000	93
276	Miami	Tadmar	"	1,128	137,300	122
277	Scioto, Upper	Columbus	"	1,032	83,500	80.8
278	Stillwater	Little York (below)	"	657	85,400	130
279	Mad	Osborn	"	649	75,700	117
280	Stillwater	W. Milton	"	611	86,200	141
281	Olentangy	Columbus	"	520	67,400	130
282	Mad	Springfield	"	505	55,400	110
283	Stillwater	Sugar Grove	"	448	51,400	115
284	Twin Cr.	Germantown	"	270	66,000	244
285	Stillwater	Greenville Cr. (above)	"	223	33,100	148
286	Ludlow Cr.	Ludlow F. (above).	"	65	17,300	266
287	Tawawa Cr.	Dayton (above)	"	52	29,700	570
288	Lost	Dayton (above)	"	54	12,900	239
289	Honey Cr., E. F.	New Carlisle	"	11.8	15,200	1290
290	Turtle Cr.	Dayton	"	35	6,130	175
291	Donnels Cr.	Dayton (above)	"	27.5	4,050	147
292	Spring Cr.	Dayton	"	27	5,670	210
293			"			
294			"			
295	Columbia	Cascade Locks	Oregon	237,000	1,400,000	5.9
296	Willamette	Albany	"	4,860	218,000	45.0
297	Umpqua	Tolo.	"	1,800	86,000	47.8
298	Willamette	Jasper	"	1,450	150,000	103
299	MacKenzie	Springfield	"	960	47,000	49
300	Willamette, E. F.	Goshen	"	690	40,000	58
301	Yamhill	Sheridan	"	290	24,800	85.4
302	Willow	Heppner	"	125	36,000	288
303	Willow	Heppner	"	20	36,000	1800
304			"			
305			"			
306			"			
307	Susquehanna	McCall's Ferry	Pennsylvania	26,766	670,000	25.1
308	Susquehanna	Harrisburg	"	24,030	736,000	30.6
309	Ohio	Pittsburg	"	19,100	440,000	23
310	Susquehanna	Dannville	"	11,100	305,000	27.5
311	Susquehanna	Wilkes-Barre	"	9,810	218,000	22.2
312	Alleghany	Killanning	"	9,010	240,000	26.6
313	Susquehanna (W. B.)	Williamsport	"	5,640	188,000	33.3
314	Monongahela	Lock No. 4	"	5,430	207,000	38.1
315	Juniata	Newport	"	3,480	139,000	39.8
316.	Juniata	Newport	"	3,380	18,200	53.8
317.	Schuylkill	Philadelphia	"	1,920	99,000	51.5
318	Kiskimineatis	Avonmore	"	1,720	67,300	39.1
319	Youghiogheny	Connellsville	"	1,320	36,600	27.7

TABLE VIII—*Continued*

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
320	Clarion	Clarion	Pennsylvania	910	39,300	43.2
321	Youghiogheny	Confluence	"	782	46,100	59
322	Youghiogheny	Confluence	"	435	33,000	76
323	Stony Cr.	Johnstown	"	428	30,000	70
324	Perkiomen	Frederick	"	152	17,600	116
325	Neshaming Cr.	Below Forks	"	139	19,000	137
326	Tohickon	Mt. Pleasant	"	102	14,100	138
327	North Fork Cr	Brookville	"	97	12,000	124
328	Chester Cr.		"	62	62,000	1000
329	Conemaugh Br.	Johnstown	"	49	10,000	204
330	Darby Cr.		"	48	27,840	580
331	Crum Cr.		"	22	9,020	410
332	Ridley Cr.		"	20	15,000	750
333	Canodhocly Cr.	Nearmouth	"	13.9	4,980	359
334	Connoquenessing	Oakland	"	13.6	4,280	315
335	Mill Cr.	Erie	"	12.9	12,900	1000
336	Spring Cr.		"	11.6	3,000	259
337			"			
338			"			
339			"			
340	Flat		Rhode Island	61	7,350	120
341			"			
342			"			
343			"			
344	Broad (Car.)	Parr	South Carolina	4,570	252,000	55.2
345	Catawba	Rockhill	"	2,987	151,000	50.5
346	Broad (Car.)	99 Island Sta.	"	1,550	186,000	120
347	Pacolet	Cacolet	"	400	35,600	89
348			"			
349			"			
350			"			
351	Tennessee	Chattanooga	Tennessee	21,382	780,000	34.4
352	Little Tennessee	McGhee	"	2,470	82,000	33.2
353	Hiwassee	Reliance	"	1,180	68,000	57.7
354	Little Tennessee	Judson	"	675	57,500	85.2
355	Ocoee	McCays	"	374	24,000	64.3
356			"			
357			"			
358			"			
359	Little River	Cameron	Texas	7,010	650,000	92.3
360	Little River	Leon Junction	"	5,300	331,000	62.5
361	San Gabriel	Georgetown	"	431	160,000	371
362	Salado Cr.	Salado	"	148	143,000	966
363	San Antonio	Below San Pedro Cr.	"	85	42,500	499
364	Brushy Cr.	Round Rock	"	74.7	34,500	462
365	Pine Cr.	Paris	"	48	19,700	410
366	San Antonio	San Antonio	"	40	8,000	200
367	Boggs Cr.	Pueblo (near)	"	26	15,100	582
368	Apache	San Antonio	"	22	15,500	704
369	Martines Cr.	San Antonio	"	19.6	24,000	1223
370	Alazon Cr.	San Antonio	"	16.9	33,000	1950
371	San Antonio	San Antonio	"	41	23,700	580
372			"			

TABLE VIII—*Continued*

Item No.	Stream	Place	State	Drainage Area in Square Miles	Flood in Sec.-ft.	Flood in Sec.-ft. per Square Mile
373			Texas			
374			"			
375			"			
376	New	Radford	Virginia	2,725	173,600	63.8
377	Shenandoah, S. F.	Front Royal	"	1,570	76,800	48.9
378	James, N. F.	Glasgow	"	831	47,000	56.5
379	Roanoke	Roanoke	"	388	24,200	62.4
380			"			
381			"			
382			"			
383			"			
384			"			
385			"			
386	Cowlitz	Mossy Rock	Washington	1,170	50,800	43.5
387	Yakima	Cle Elm	"	500	33,000	66
388	Clealum	Roslyn	"	205	25,000	122
389	Cedar	Rarenosdale	"	170	15,000	88.3
390	Cedar	Rarenosdale	"	79	9,480	120
391	Naches	Niles	"	63.6	21,700	341
392	Yakima	Martin	"	56	9,800	175
393			"			
394			"			
395			"			
396			"			
397	Ohio	Wheeling	West Virginia	23,800	495,000	20.8
398	Shenandoah	Millville	"	2,995	140,000	
399	Greenbriar	Alderson	"	1,344	56,000	
400	Elkhorn	Keystone	"	44	16,000	364
401			"			
402			"			
403			"			
404			"			
405	Laramie Reservoir Outlet	Laramie	Wyoming	72	7,000	97
406			"			
407			"			
408			"			
409			"			

From a study of similar records, many empirical equations for flood flows have been proposed. Ten of the best known of these equations are given in Mead's "Hydrology,"[6] p. 580. These equations have been devised mostly in connection with studies for particular localities or for a particular character of drainage area and are not applicable to general conditions. Mead's diagrammatic comparison of these equations, showing extreme variations in results, indicates that they are inapplicable to flood-flow studies unless the user is intimately acquainted with their limitations and the methods of their derivation.

[6] McGraw-Hill Book Co., 1919.

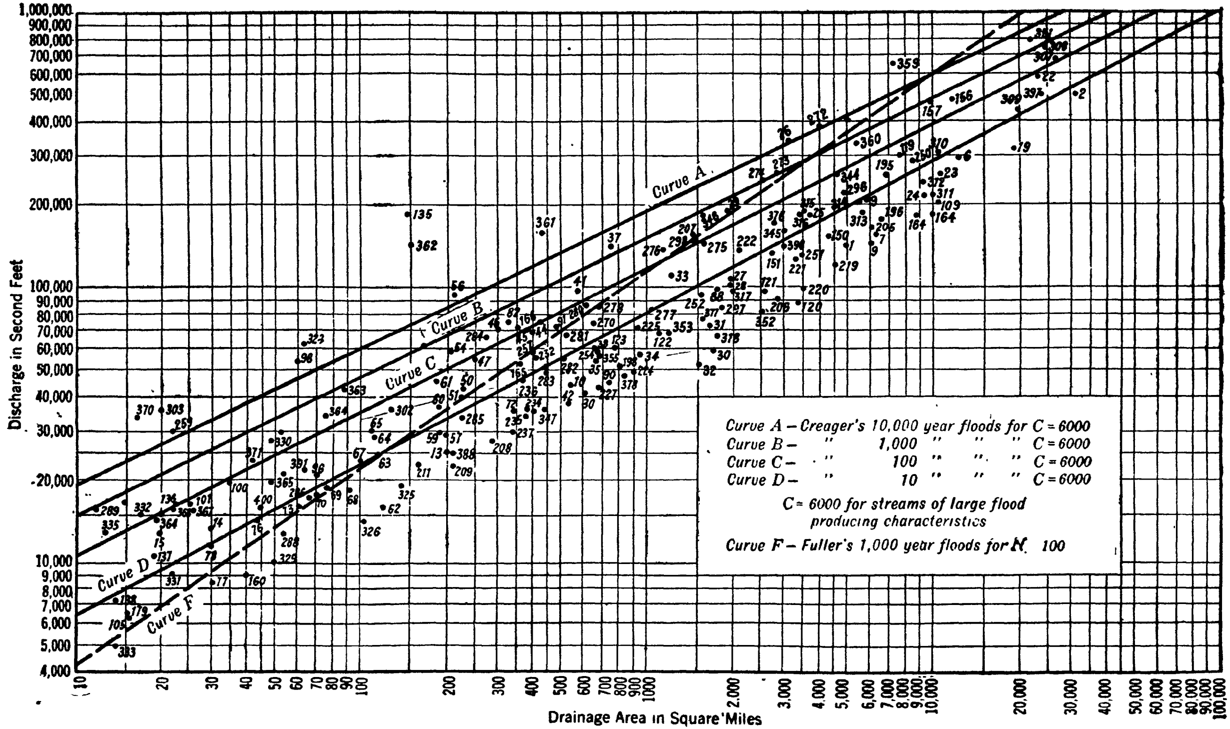

FIG. 28.—Unusual Flood Discharges, United States Rivers.

The most recent of these equations is that proposed by Fuller.[7] His equations for flood probabilities are:

$$Q_A = NA^{0.8}(1 + 0.8 \log T), \quad \ldots \quad (1)$$

$$Q = Q_A(1 + 2A^{-0.3}). \quad \ldots \quad (2)$$

Therefore,

$$Q = NA^{0.8}(1 + 0.8 \log T)(1 + 2A^{-0.3}), \quad \ldots \quad (3)$$

where Q_A = the maximum twenty-four-hour average flood, in cubic feet per second, likely to be equaled or exceeded on an average of once in T years;

Q = the corresponding momentary peak flow;

N = a coefficient which is constant for each watershed, and depends on its characteristics;

A = the area of the watershed in square miles.

Fuller's Equation (3), for $T = 1000$ years and $N = 100$, is shown in Fig. 28 as Curve F. Additional flood data have been published since Fuller's equation was derived, and his curve does not follow closely the general trend of the points plotted in Fig. 28. The author has therefore drawn Curves A, B, C, and D. It is considered that all floods above Curve A are at least 10,000-year peak-flow probabilities on streams having characteristics most favorable to large floods. This does not seem unreasonable if it is remembered that the recorded floods above the curve are the only floods of such magnitude reported from thousands of closely watched streams during a long period of years. The isolated extreme floods indicated in Fig. 28 are plotted in accordance with published reports; but, as the facilities in each instance were not always favorable to close measurement, it is considered that they may be questionable.

The equation of Curve A is:

$$Q' = 6000A^{0.5}. \quad \ldots \quad (4)$$

Or, in second-feet per square mile:

$$q' = \frac{6000}{A^{0.5}}. \quad \ldots \quad (5)$$

This equation being used to represent 10,000-year probabilities on streams favorable to large floods, Curves B, C and D, representing similar 1000-, 100- and 10-year probabilities, were estimated from the average relation of 10,000-year floods to floods of smaller periods, as indicated by a study of many frequency curves of flood discharges. This relation was found to agree closely with the relation for rainfall probabilities, indicated in Fig. 13. The author's resulting flood-frequency equation is:

$$Q = CA^{0.5}\left[\frac{2 - e^{-0.04A^{0.3}}}{3}\left(1 - \frac{\log 0.1T}{3}\right) + \frac{\log 0.1T}{3}\right], \quad \ldots \quad (6)$$

[7] Trans. Am. Soc. C. E., Vol. LXXVII, p. 564.

where, in addition to the preceding notation,

e = the base of the natural system of logarithms. Log $e = 0.4343$;
C = a coefficient depending upon the characteristics of the drainage area, or 6000 for areas having characteristics most favorable to large floods.

For $T = 10{,}000$ and $C = 6000$, Eq. (6) becomes Eq. (4). Curves for four different values of T and for $C = 6000$ have been plotted in Fig. 28. That more floods above the 10-year flood, Curve D, have not been recorded is attributed to the fact that 10-year floods are not reported as unusual floods. Many of the floods above and below Curve D are 100- to 10,000-year probabilities on streams having flood coefficients, C, much less than 6000.

No method has yet been devised by which the choice of flood coefficient, C, can be determined from the physical characteristics of the watershed except by the judgment of the engineer, confirmed by other methods of investigation. Until the various characteristics affecting flows have been better coordinated by many more years of records and investigations, the choice of coefficient, without runoff records, is likely to be considerably in error, and ample factors of safety should be provided. Perhaps the best indication of the flood coefficient can be obtained from a study of the frequency curves of Sec. 33.

36. Comparison with other Rivers.—A knowledge of the physical factors affecting the magnitude and frequency of floods is essential in the investigation of flood flows, particularly if the period of stream gagings at or near the site is not of sufficient length to permit of accurate frequency studies, and if comparisons are desired with other streams on which the flood tendencies are known. Aside from differences in area of the watershed, two streams may have materially different flood tendencies, accounted for by a difference in the characteristics of the watershed. The flood coefficient, C, of Sec. 35, the use of which is made necessary by such differences in characteristics, depends on three general conditions, there being several subdivisions, as explained later. These conditions are as follows:

(1) The prevailing conditions of rainfall;
(2) The storage capacity of the watershed, or its ability to retain temporarily, above or below ground, and to distribute, the maximum rainfall;
(3) The capacity of the watershed to release stored waters suddenly.

The average annual rainfall on a watershed is not an indication of the maximum rate or intensity of precipitation which may be expected. This is well illustrated by a comparison of Fig. 1, showing isopluvials of mean annual rainfall in the United States, which run approximately north and south, with Fig. 12, showing lines of equal maximum rainfall intensities, which run generally east and west.

Storage, of whatever nature, has a tendency to reduce the size of floods. The storage capacity of the watershed may be divided into the following items:

(1) Storage in reservoirs, lakes, and swamps;
(2) Storage below the ground surface;
(3) Storage above the ground surface.

It is seldom that storage *below* the flow line of artificial reservoirs is effective in reducing the peak of large floods, because at such times the reservoirs are likely to be full, owing to the excessive flow preceding the peak. Storage *above* the normal flow line of all bodies of water is always available, as such bodies of water must suffer an increase in surface elevation to provide sufficient head and area at the outlet to accommodate the increasing flow.[8] The percentage of area of reservoirs, lakes, and swamps has considerable influence on the value of the flood coefficient.

The magnitude of floods is always less on rivers draining deep sandy areas, in which the storage below the ground is considerable. If such areas are large and extend to the higher elevations of the watershed, the effect on floods may be considerable.

Storage above the ground is affected by the nature of the vegetation, the shape and slope of the drainage area, and the characteristics of the river bed and banks. It is evident that those characteristics which will permit of rapid runoff of the precipitation to the site of the dam will result in large floods. Rocky slopes, devoid of vegetation, are conducive to quick discharge. Conversely, areas covered with dense vegetation will prove effective in holding back the water and smoothing out the peak of the flood. Heavy underbrush is particularly effective in this respect, as the rivulets are held back by friction in passing around and among the stalks of the plants and such branches as have fallen or have been beaten down to the ground surface. Practically no water, at the peak of the precipitation, is held back by adherence to leaves and branches above the ground surface. For this reason many engineers are of the opinion that it is the removal of the dense underbrush, rather than that of the large trees, which has increased flood tendencies in districts that have been deforesed.

Steep slopes will, of course, produce rapid runoff. Therefore, floods from mountainous districts are relatively severe.

In rivers having tributaries extending in the shape of a fan from a given point, and of approximately the same size, the peak of the flood from each of the tributaries is likely to reach the main stream and the dam at approximately the same time, resulting in relatively large floods. Conversely, when the catchment area is relatively narrow, with tributaries of different sizes discharging into the main stream at regular intervals, the peak of the runoff from the tributary areas will reach the dam at different times, resulting in relatively small floods. A large number of tributaries is also productive of rapid runoff.

Rivers and tributaries that have frequent restricted places and rough bottoms, and are relatively shallow in comparison with their widths, may also be said to have a moderately retarding effect on the rapidity of runoff.

The capacity of the catchment area to release stored water suddenly may be indicated by:

(1) The frequency and magnitude of ice and log jams, with consequent danger of release of impounded waters at or near the peak of the flood.

[8] See Sec. 20 and Sec. 37.

(2) The presence of other dams of questionable strength, impounding large volumes of water. A number of well-designed dams have failed on account of the destruction of defective dams above, with a resultant enormous increase in the runoff due to the sudden release of stored waters.

(3) Temporary partial blocking of the flow of the stream, due to the lodgment of debris against submerged bridges, and subsequent failure of the bridges, with a release of the impounded waters at the critical time.

(4) Storage in the form of snow, which may be suddenly released by a record precipitation accompanied by a rise in temperature.

It is probable that, in general, the area of the watershed, the maximum rate and duration of rainfall, the steepness of the slopes, the geological formation, the slope of the drainage area and arrangement of tributaries, and the nature of the vegetation will, in the order given, have the most influence on the flood tendencies of the stream. The items mentioned as affecting the capacity of the stream to release stored waters suddenly cannot be included in a general classification, as their effect on floods is too uncertain.

The flood characteristics of two streams may be compared as illustrated in Fig. 29, which indicates the relation between simultaneous floods on the Wallkill River and Rondout Creek, N. Y., adjacent watersheds.

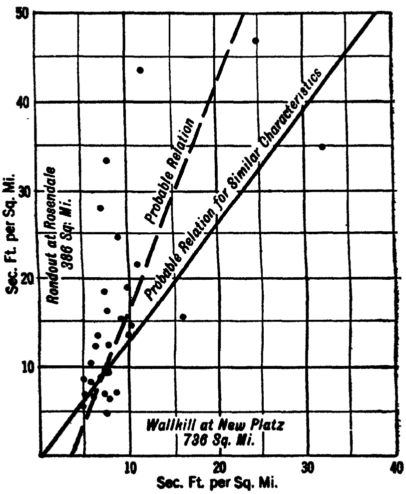

FIG. 29.—Comparison of Flood Flows, Wallkill River and Rondout Creek.

From Eq. (5) of Sec. 35, the flood runoff per square mile from the 736 square miles of the Wallkill watershed would ordinarily be about 78 per cent of that from the 386 square miles of the Roundout watershed, other conditions being the same. This theoretical relation is indicated by the full line in Fig. 29. Actually, however, the probable relation is a line drawn through the plotted floods, as indicated by the dash line, showing that the Rondout Creek has greater relative flood tendencies than the Wallkill. The Walkill watershed has flatter slopes and a greater extent of swamps. Small floods on the Walkill are, however, relatively larger than on the Rondout, which is accounted for by the fact that a small artificial reservoir just above Rosendale smooths out the smaller floods but has no appreciable effect on the large floods.

Relations of this kind furnish a means by which the flood characteristics of a stream may be obtained approximately with relatively short periods of runoff records, if long-term records are available on a neighboring stream.

37. Effect of Artificial Storage on Floods.—The storage provided above the flow line of large storage reservoirs on small streams, as the water rises above the overflow, affects a considerable reduction in the magnitude of flood flows on the stream. It is not safe to rely on storage below the flow line, as the reservoir may be full just before the flood. The data required for routing a flood past a dam are:

(1) An inflow hydrograph of the flood;

(2) A rating curve of the spillway;

(3) A storage curve of the spillway.

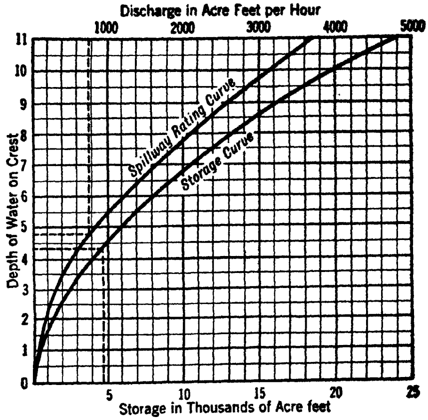

FIG. 30.—Typical Rating and Storage Curves.

The units acre-feet per hour for discharge and acre-feet for quantity are usually adopted for routing studies, instead of second-feet and cubic feet, as the former units are more convenient.

The character of the inflow hydrograph must be determined from a study

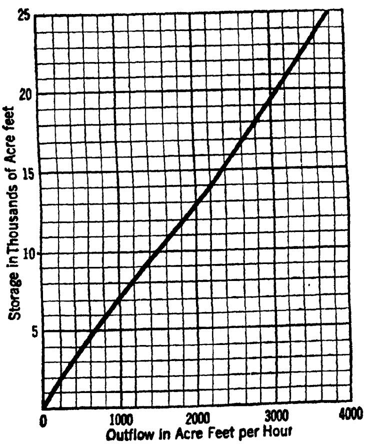

FIG. 31.—Typical Storage—Outflow Curve.

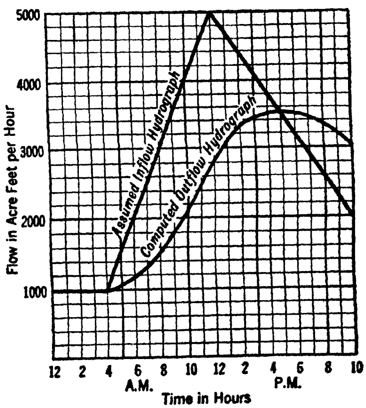

FIG. 32.—Typical Regulation by Storage Above the Flow Line.

of hydrographs of past floods. It is customary to adopt straight lines for the inflow hydrograph in consideration of the approximate nature of the data used. The inflow hydrograph and rating curve of the spillway in the following example are indicated in Figs. 32 and 30 respectively. The storage curve,

showing the total storage above the crest of the dam, is computed from surveys of the reservoir area and plotted as shown in Fig. 30.

These data being in hand, the storage-outflow curve of Fig. 31 is plotted by combining the two curves of Fig. 30. Figure 31 shows the relation between simultaneous total storage above the crest and outflow, both being measured by depth of water on the crest.

Probably the easiest and best method of routing floods is the trial-and-error step method. More exact methods have been published;[9] but it is considered that the shorter and less exact method used in this example is well within the accuracy of present hydrological knowledge.

In the following example, it is assumed that, just before the flood starts, the water surface in the reservoir is stationary and the outflow is 1000 acre-ft. per hour, corresponding to 5.5 ft. on the crest.

Assume increments, t, of time during which the rates of change of inflow and outflow are approximately constant. A period of two hours is adopted for this example. The necessary calculations are indicated in Table IX. From Fig. 32, the inflow at the beginning of the first period, or at 4 A.M., is 1000, and at the end of the two-hour period, or at 6 A.M., it is 2000 acre-ft. per hour. The average inflow during the period is therefore 1500 acre-ft. per hour, and the total inflow for the two hours is 3000 acre-ft. This is all tabulated in the first column of Table IX.

At this stage, a guess is made at the outflow at the end of the first period. (For the intervals after the first, the guess is influenced by the general direction of the outflow curve which is being plotted.) If this is assumed to be 1200 acre-ft. per hour, and the outflow at the beginning is known to be 1000 acre-ft. per hour, the average outflow during the period is therefore 1100 acre-ft. per hour and the total outflow for the two-hour period is 2200 acre-ft.

Since, during the period, the inflow was 3000 acre-ft. and the outflow 2200 acre-ft., the difference, or 800 acre-ft., must be taken by storage above the crest. The accumulative storage at the beginning of the period was 7000 acre-ft. and, therefore, at the end of the period, it is 7800 acre-ft.

Now, referring to Fig. 31, it is seen that 7800 acre-ft. corresponds to less than the outflow of 1200 acre-ft. per hour, tentatively adopted. The calculations must, therefore, be repeated with a new assumption of outflow, as shown in the second column of Table IX, in which 1150 was adopted and 7850 acre-ft. storage calculated. This agrees nearly enough with Fig. 31 and is assumed to be correct. The succeeding intervals are calculated in the same manner, only the last trials of each period being indicated in the table. The resulting outflow hydrograph is plotted in Fig. 32.

It should be noted, as a check on the calculations, that the outflow and inflow curves of Fig. 32 always cross at the highest point of the outflow curve; also that the area between the outflow and inflow curves is equal to the total storage utilized, in this case being equal to 22,520 − 7000 = 15,505 acre-ft. It is seen that, in this case, the 5000 acre-ft. per hour flood was reduced 30 per cent by virtue of the storage above the flow line.

[9] See Reference 2, Sec. 38.

TABLE IX

TYPICAL CALCULATIONS FOR ROUTING FLOOD

(Intervals in Hours)

	First Trial	Last Trial	Last Trials							
	4–6	4–6	6–8	8–10	10–12	12–2	2–4	4–6	6–8	8–10
1. Inflow at beginning, acre-feet per hour	1000	1000	2,000	3,000	4,000	5,000	4,400	3,800	3,200	2,600
2. Inflow at end, acre-feet per hour	2000	2000	3,000	4,000	5,000	4,400	3,800	3,200	2,600	2,000
3. Average inflow, acre-feet per hour	1500	1500	2,500	3,500	4,500	4,700	4,100	3,500	2,900	2,300
4. Inflow for t hours, acre-feet	3000	3000	5,000	7,000	9,000	9,400	8,200	7,000	5,800	4,600
5. Assumed outflow at end,* acre-feet per hour	1200	1150	1,560	2,130	2,800	3,300	3,500	3,500	3,350	3,100
6. Outflow at beginning,† acre-feet per hour	1000	1000	1,150	1,560	2,130	2,800	3,300	3,500	3,500	3,350
7. Average outflow, acre-feet per hour	1100	1075	1,355	1,845	2,465	3,050	3,400	3,500	3,425	3,225
8. Outflow for t hours, acre-feet	2200	2150	2,710	3,690	4,930	6,100	6,800	7,000	6,850	6,450
9. Storage required,‡ acre-feet	800	850	2,290	3,310	4,070	3,300	1,400	0	− 1,050	− 1,850
10. Storage stage at beginning,§ acre-feet	7000	7000	7,850	10,140	13,750	17,820	21,120	22,520	22,520	21,470
11. Storage stage at end,¶ acre-feet	7800	7850	10,140	13,450	17,820	21,120	22,520	22,520	21,470	19,620

* To be checked from line 11 and Fig. 31.
† From line 5 of previous interval.
‡ From line 11 of previous interval.
§ Inflow from line 4, less outflow from line 8.
¶ Sum of lines 9 and 10.

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CHAPTER VI

CAPACITY OF THE DEVELOPMENT

By William P. Creager

39. General.—Many factors enter into the determination of the most economical capacity of installation for a water-power development, and of the extent of storage and pondage that must be created in order to utilize, to the greatest economical extent, the flow of the stream. Most of these factors are so interrelated that the problem has no direct solution. For important, complex developments, many studies must be made, each involving different assumptions of development arrangement, before a final scheme can be adopted. Figure 33 is a chart showing in a general way the various factors governing the choice of development arrangement and the determination of station output. These factors and the usual order of study, are outlined briefly here, but will be described in more detail in succeeding sections.

In determining the capacity of the development (9), it is necessary to consider, jointly, the market requirements (1), the head available at the site (2), the natural flow of the stream (3), the extent of possible economical regulation of the flow (4), the probable cost of the development (5), the value of the output (6), the nature and capacity of existing and proposed auxiliary power plants (7), and the extent to which the plant is to be installed to carry peaks of inadequately ponded plants (8). Perhaps the most salient influence affecting the development scheme is the class of power to be produced. Two general cases are given below.

Case I. For Primary Power.—As pointed out in Sec. 30, in which the functions of storage and pondage were described, the variations in natural stream flow correspond in no degree to the variations in power demand. Therefore, if the market requires an uninterrupted, or primary power output, and no storage, pondage, or auxiliary power plant is to be provided, the capacity of the development must be limited so as to correspond to the minimum natural flow of the stream, because there must always be water available when the station is called upon for full output.

If adequate pondage is to be provided, the capacity must be limited to an amount such that the average output (which is usually much less than maximum output) corresponds to the minimum flow of the stream.

If storage is available but no pondage is to be provided, the capacity must be limited to correspond to the minimum regulated flow of the stream.

If both storage and pondage are to be provided, the capacity must be lim-

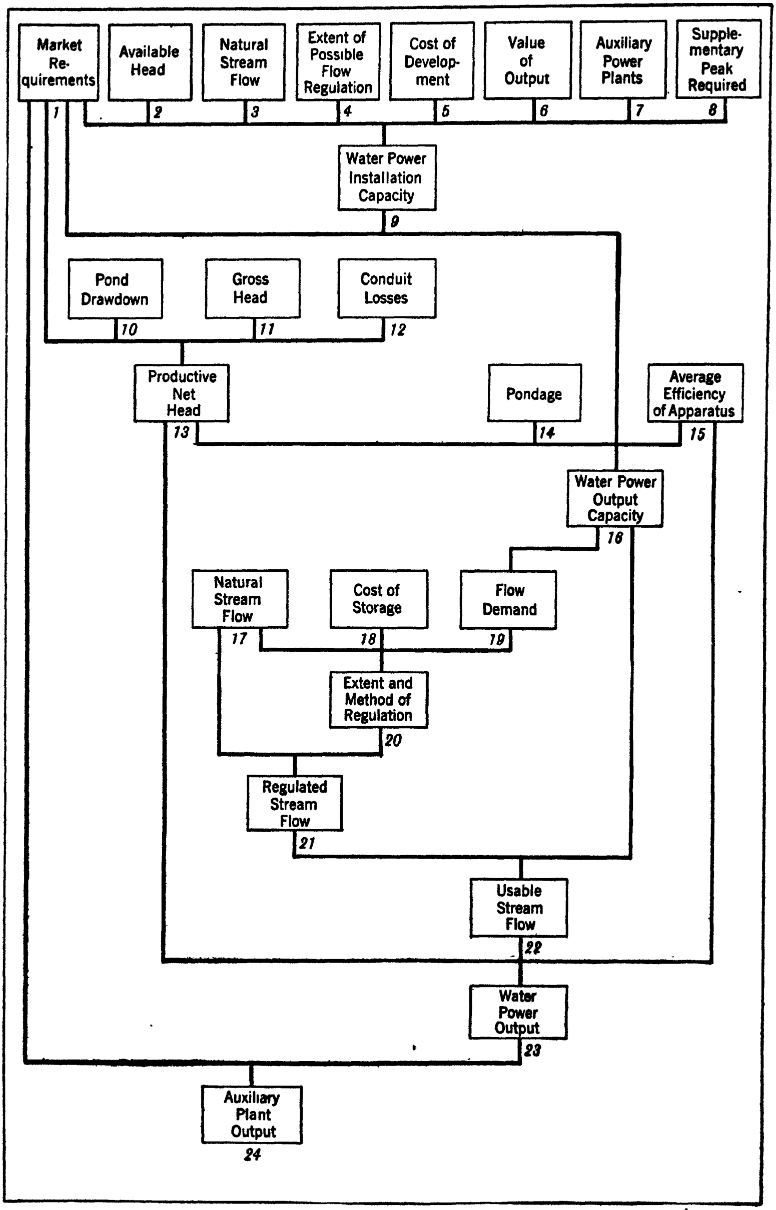

Fig. 33.—Chart Showing General Factors Affecting Plant Capacity and Output.

ited to an amount such that the average output corresponds to the minimum regulated flow.

In all cases, the capacity of the development may be increased to the extent that auxiliary power can be provided to supply the deficiencies of output due to lack of natural or regulated flow.

Case II. For Secondary Power.—If the nature of the market is such that a part of the output can be sold as secondary power, that is, if the power can be interrupted during periods of low stream flow, the capacity of the development can be increased to any desired extent.

Thus it is seen that, for primary power output with no auxiliary plant, the capacity of the development is definitely fixed by the amount of minimum natural or regulated stream flow. For primary power with an auxiliary plant to supplement deficiencies in output, and for secondary power, the capacity is limited only by the necessity of producing power at a cost which will prove profitable. The cost of the development, per kilowatt of installed capacity, decreases as the capacity of the installation increases, because there are many constant items of cost, as dam and real estate, which are not dependent upon the extent of the installation. On the other hand, the energy output per kilowatt of installed capacity decreases as the capacity increases, because the larger the flows utilized for power the smaller the percentage of time they are available. Hence there is a definite limit to the capacity, beyond which the output cannot be produced at a profit. This limit depends mainly upon the cost per kilowatt of installation and the value of the output. The cost of reservoirs, of course, enters directly into the determination of the extent to which storage and pondage can be created to regulate the natural flow.

Many of the factors influencing the choice of plant capacity may have to be approximated, subject to later verification. For example, the most economical plant capacity (9) depends upon the extent of possible flow regulation (4); but, in the usual order of study as shown in Fig. 33, the latter cannot be exactly determined until after the plant capacity has been fixed.

On streams without storage, it is not uncommon, for developments supplying large blocks of secondary power or supplementing steam plants, to install up to the flow that may be expected only 30 per cent of the time as shown by a duration curve.[1] In such cases the installation must be relatively inexpensive or the output quite valuable, as parts of such an installation must lie idle a large part of the time. It is seldom that a water power, without storage, can be developed profitably for primary power only, unless supplemented by other power, because the unregulated minimum flow of most streams is practically negligible compared with the flow required for practical operation.

Capacity of installation, in excess of that required to meet the peak demand, is frequently required. As capacity cannot be installed in small increments, considerable excess capacity may be required to accommodate a gradually increasing market. If the development is being operated independently and without auxiliary standby stations, spare units may be required

[1] Fig. 55.

to safeguard the output during periods of shut-down of a unit for repairs or maintenance. Shut-downs are quite infrequent in large modern stations and spare units are seldom required if the plant is to be operated for secondary power. The advantage of operating on a large system, where a single spare unit may be made to serve all the plants on the system, is quite obvious.

Capacity in excess of that shown to be profitable for the plant itself may be installed if such excess capacity can be shown to supplement advantageously other plants on the system which have no ponds. Excess capacity for this purpose can be used only in plants having ample pondage. Such plants are termed " peak-load plants " because they, having pondage, can carry most of the peak demands at a low load factor while other plants having little or no pondage carry the base of the load demand at a high load factor.

The capacity of the development (9) having been fixed, it is usual to establish what is known as the water power output capacity (16). This consists simply in a relation between stream flow and usable flow. It is frequently set forth as an equation or in the form of a diagram which shows, for any given discharge, the amount of such discharge that can be converted into useful work. Such a diagram is given in Fig. 46 and further described in Sec. 51. As indicated in Fig. 33, the output capacity (16) depends upon the capacity of the installation (9), the market requirements (1), the average efficiency of the apparatus (15), the pondage (14), and the head (13).

The stream flow (21), regulated or unregulated as the case may be, is then set forth in tabular or diagrammatic form, and the usable amount of such flow (22) determined from the water power output capacity relation (16), and converted, according to the head (13) and efficiency (15), into total water power output of energy (23) for a given period. The difference between the water power output (23) and the market requirements (1) must be supplied by auxiliary plants (24) if primary power is required.

The net head at the development and the " productive net head " (13), described in Sec. 49, are dependent upon the gross head (11), the pond drawdown (10), and the friction losses in the conduits (12).

The flow demand (19) is the discharge required to meet all power demands (16) and it is desirable to have as much as possible of this flow continuous. With the flow demand (19) as the objective, the extent and method of economical regulation (20) depend upon the amount and variation in the natural stream flow (17) and the cost of storage reservoirs (18).

The regulated stream flow (21) is the natural stream flow (17), altered to the greatest extent by the adopted storage (20).

40. Market Requirements.—The power requirements vary with the nature of the market to be served. The three principal uses for power are for manufacturing, lighting, and traction. Each has distinguishing characteristics but they vary considerably in different localities and during different periods.

Market requirements are usually shown by load curves, in which power demand is indicated by ordinates and time by abscissae. A typical summer, ... y, city load curve is shown in Fig. 34.

The manufacturing load starts in heavily about 7 A.M. and, on week days, continues until about 6 P.M. with a short reduction in load at the lunch hour. The heavy lighting load comes on at dusk. The morning traction load is greatest between 6 and 8. The highest traction peak usually occurs after the factories close down in the afternoon. The total load is also indicated. The summer peak load likely to occur at any time during the day, in this case at 9 A.M.

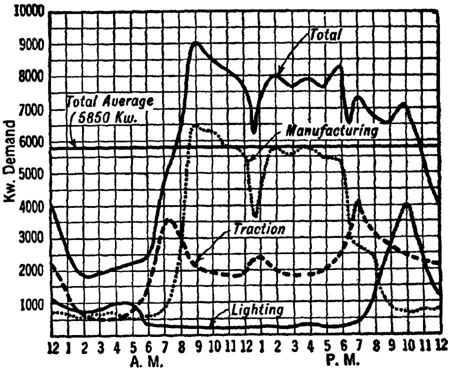

FIG. 34.—Typical Summer Week Day Load Curve for City Distribution.

A typical winter, week-day load curve is shown in Fig. 35. It will be noted that, during the winter, the lighting and manufacturing loads overlap, resulting frequently in the greatest demand during the late afternoon or early evening. The maximum yearly peak usually occurs during the week before Christmas, when lighting and traction demands are greatest.

A typical summer weekly load curve is indicated in Fig. 36. The week-day curve is a duplicate of Fig. 34, but in this case average hourly demands are indicated. In this district, most of the factories are closed on Saturday afternoons and all day on Sundays, resulting in a reduced power demand during those periods. The shaded portions of the curve are used to indicate required pondage as described in Sec. 51.

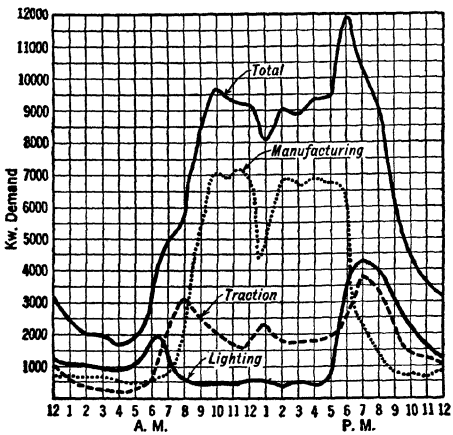

FIG. 35.—Typical Winter Week-Day Load Curve for City Distribution.

Power output is usually recorded in the power station log as average hourly demands, and is customarily so indicated in load curves, as shown in Fig. 36. Unless, as is quite unusual, the pond is insufficient to regulate the stream flow to accommodate the varying hourly demands, the actual peak during the hour is not of particular interest except during the hour of maximum yearly demand, which fixes the capacity of the installation to provide the peak load. The ratio of the yearly peak load to the corresponding average hourly load varies from

about 5 per cent in very large systems to as much as 25 per cent or more on small systems and for traction and other extremely fluctuating loads.

Many pumping loads are virtually continuous during periods of operation. Deep mine pumping is usually practically constant throughout the year. Shallow mine pumping varies with the changing ground flow. Pumping for municipal water supply from a low source may be confined to periods of low water in the main gravity system, which is supplied from a high source. In such cases the load is not suitable for unregulated water powers supplying other markets, because such periods correspond to reasons of deficiency in flow available for power. The same is true for irrigation pumping loads, water

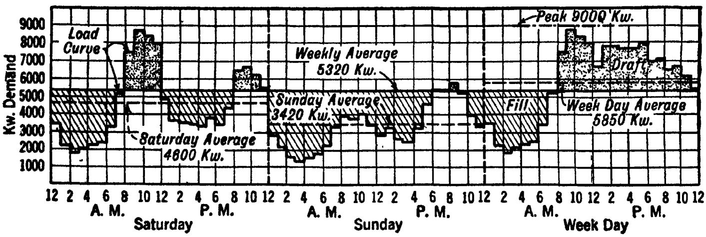

FIG. 36.—Typical Summer Weekly Load Curve.

being required for irrigation only during the dry season when unregulated flow for power is small.

Seasonal changes in general demands may be caused by normal variations or by permanent changes in market conditions. The normal seasonal variations in pumping loads have been previously described. Lighting loads are always greater during the dark days of winter. In summer, Sunday traction loads may exceed week-day loads if outlying pleasure resorts are popular. Seasonal variations in manufacturing loads depend largely upon the nature of the industries. Many operations, such as the working of the quarries and surface mines in cold climates, are suspended during the winter season. On the other hand, wood-pulp mills are usually arranged to grind extensively during the wet season in order to store sufficient pulp for use during dry months when the flow is sufficient only for paper machines. In general, however, city distribution demands are slightly less during the summer. A typical annual load curve for a general city load is indicated in Fig. 37. This curve shows clearly the permanent changes in market conditions as indicated by the increasing annual demands, which require frequent additions to the power system to keep pace with the growing market.

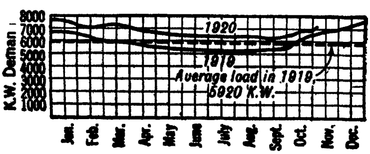

FIG. 37.—Typical Yearly Load Curve Having Low Summer Demand.

Estimates of market requirements, to be satisfied at and after the completion of the development, are usually based on the rate of growth during

previous years, although special knowledge of large future customers is sometimes available. In many instances provision is made in foundations, headworks, and other parts of the development, for the future installation of additional apparatus to supply the predicted market requirements of future years, and studies of development arrangement should always include this possibility.

The following definitions have been adopted for the Standards of the American Institute of Electrical Engineers, 1921:

The *load-factor* is the ratio of the average power to the peak power. In each case, the interval of maximum load and the period over which the average is taken should be definitely specified, such as a " half-hourly monthly " load factor. The proper interval and period are usually dependent upon local conditions and upon the purpose for which the load factor is to be used.

It will be shown later that it is quite important, in the use of the load factor, to know where the power is measured. In the case of hydro-electric developments, it is customary to measure the power at the low-tension side of the substation transformers. The average power is taken over a certain period of time, such as a day, a month, or a year, and the maximum is taken as the maximum load within that period. The maximum load is sometimes the five-minute maximum peak, half-hourly maximum peak, or hourly maximum peak; but in the case of hydro-electric developments the momentary or instantaneous peak should be used because the installation has no excess over rated capacity as in the case of steam plants.

The momentary daily load factor during the day, indicated in Fig. 34, is 65 per cent, the average load being 5850 kw. and the momentary peak 9000 kw. For the whole week indicated in Fig. 36, the hourly weekly load factor is 61.2 per cent, the average being 5320 kw. and the hourly peak 8700 kw. The momentary weekly load factor is 59.1 per cent, the average being 5320 kw. and the momentary peak 9000 kw. In Fig. 37 the average 1919 yearly load is shown to be 5920 kw. As the momentary yearly peak in 1919 was 12,900 kw., the momentary yearly load factor was 45.9 per cent. Thus the load factor should be qualified by the period it is intended to cover, as " the momentary summer week-day load factor," " the hourly winter weekly load factor," " the momentary yearly load factor," etc. Unless so qualified, the term is intended to mean the momentary yearly load factor, which may also be the momentary weekly load factor if the weekly output and peaks are constant throughout the year.

The load factor of small public service developments is often as low as 20 or 30 per cent. By a proper choice of customers using power for diversified purposes, the load factor for medium and large plants may be built up to 80 per cent or even 90 per cent. A common load factor for medium-sized city systems is from 40 to 60 per cent.

The load factor is an indication of the operating efficiency of the development, or the extent to which the total installation is being utilized.

The combination of two or more power systems invariably results in an increase in load factor, because the load factor increases with the number of connected customers.

The demand factor of any system, or part of a system, is the ratio of the

maximum demand of the system, or part of a system, to the total connected load of the system or of the part of the system under consideration.

In most cases the total capacity of all the motors, lights, and other uses for power in a given industry or part of the system is never used simultaneously, and therefore the demand is usually less than the connected load. If the total connected load is 100 kw. and the required delivered peak is only 80 kw., the demand factor is 80 per cent.

The diversity factor of any system, or part of a system, is the ratio of the sum of the maximum power demands of the subdivisions of the system, or part of a system, to the maximum demand of the whole system, or part of the system, under consideration measured at the point of supply.

Some engineers prefer to use the reciprocal of this ratio. For the first definition, the diversity factor is always greater than unity and for the second, less than unity. If three parts of a system have maximum demands of 100, 200, and 300 kw. respectively, the sum of these demands is 600 kw. The maximum demand of the combined three parts will be less than 600 kw. because the three maximum demands will not occur simultaneously, and if it amounts to 400 kw. the diversity factor will be 1.5 or $66\frac{2}{3}$ per cent according to whether the first or second definition is used.

The plant factor (or *capacity factor*) is the ratio of the average load to the rated capacity of the plant.

As used hereinafter, the average load for determining plant factor is the average output of the generators measured at the switchboard. The rated capacity of the plant should be the rated capacity of the turbines, measured at the switchboard.

The chart given in Fig. 38 will serve to illustrate the use of the foregoing factors in estimating the installation and average output required to serve a given market. It is very difficult to estimate accurately the various factors unless the entire market has been thoroughly metered.[2] Great care must be taken in the use of published data regarding these factors as their meaning is not always the same as in the definitions given by the American Institute.

If, to provide a spare unit or to provide for future growth, the installation for the foregoing example were 10,000 kw., the plant or capacity factor would be $\frac{3280}{10,000} = 32.8$ per cent.

It will be noticed that the ratio of the average power output of the generator to its peak output is

$$\frac{3280}{8330} = 39.4 \text{ per cent.}$$

and the ratio of the average delivered power to the peak delivered power is

$$\frac{3020}{6660} = 45.4 \text{ per cent.}$$

[2] See References 2, 3 and 4 of Sec. 46.

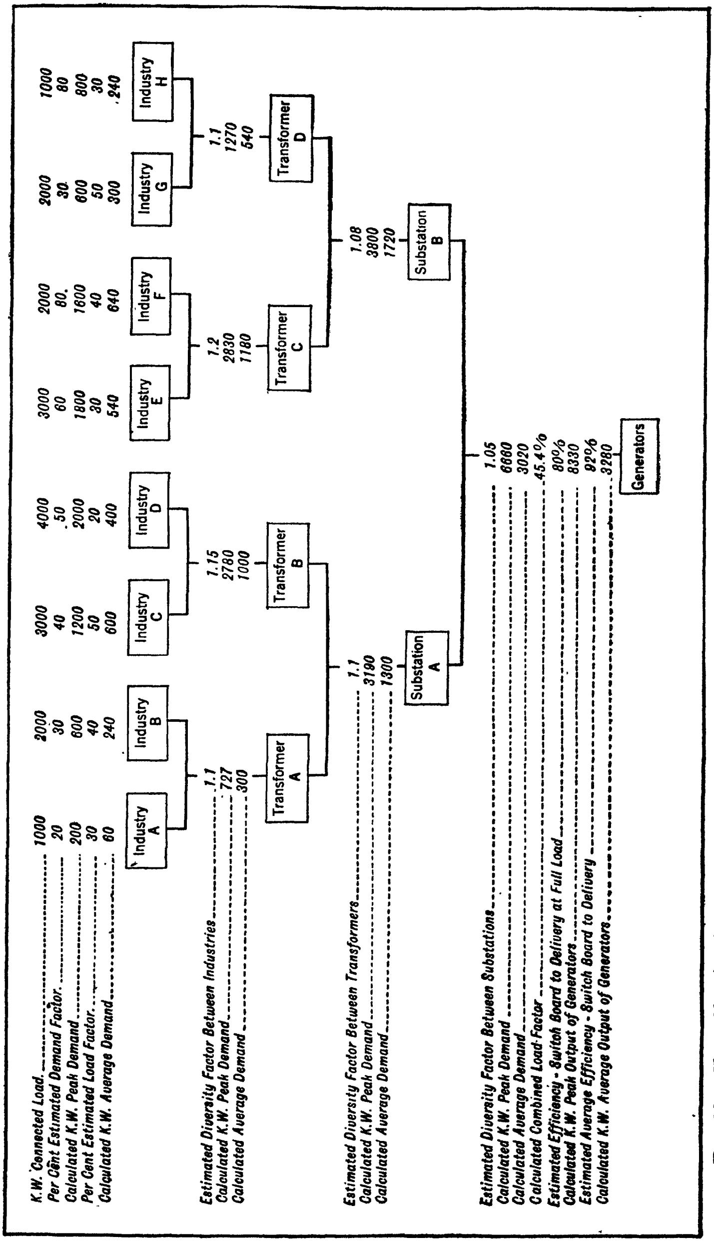

Fig. 38.—Chart Showing Sample Calculations for Estimated Average and Peak Output of Generators from Market Studies.

The first corresponds to the load factor if the power is measured at the switchboard, and the second if it is measured at the point of delivery. The difference is accounted for by the fact that the efficiency of the station is different for full and average loads. Thus, in the use of load factor, the importance of knowing where the power is measured is clearly brought out.

The total output is delivered for a diversity of uses which vary in importance according to the terms of the power contracts. That portion of the power which must be maintained equal to the demand at all seasons is termed "primary power." Those loads which may be discontinued during periods of low stream flow are classed as "secondary-power" loads. Primary power loads are sometimes of such great importance that duplicate transmission lines, spare power units and auxiliary plants, not otherwise needed, are required as safeguards against the severe penalties which a shutdown would involve.

For primary power, all deficiencies in output due to insufficient stream flow must be supplied by an auxiliary plant. Auxiliary plants may be stations using steam or other fuel or they may consist of other water powers provided with adequate storage. Power is sometimes purchased from other companies for this purpose.

Secondary-power loads are usually quite undesirable from the consumer's standpoint and such loads command a much lower selling price than primary-power loads. Frequently, unregulated water powers are developed as an adjunct to large steam-power stations to conserve fuel during seasons of high water and to make more uniform, and hence more economical, the steam-plant operation during periods of low water. In the latter case, an adequate pond is essential. The total low-water flow is stored in the pond and used during the hours of great load demand to take the peaks as described more in detail in Sec. 51.

41. Available Head.—The probable net head on the development must be fixed tentatively for studies of its influence on the choice of plant capacity. Unless the extent to which a stream may be developed for power is definitely limited by ownership of lands, railroads, bridges, or other plants, it is seldom possible to fix closely the most economical head, as a rigorous investigation requires a comparison of results from a number of different heads.

There is practically no limit to the head which may be economically developed, those of existing plants ranging from several feet to several thousand feet.

42. Possible Flow Regulation.—For primary-power installations and even for secondary-power plants, storage on small and medium-sized streams is often a very profitable investment. For primary-power output, the minimum flow may often be increased five or ten times with relatively small storage. In some instances storage on small streams has been provided to regulate practically the whole flow of the stream. Storage on large streams is seldom economically possible as the amount of water involved is usually beyond practical control. Sufficient pondage to regulate a week's flow to suit the varying weekly demand is usually indispensable to a successful development. Fortunately, it is seldom difficult to obtain sufficient pondage at little expense;

in fact, in most cases, the pond created by the diversion dam is ample for this purpose with relatively small drawdown.

43. Cost of Development.—Few reliable statistics of the cost of water powers, per kilowatt of installed capacity, are available. Published data in many instances fail to record the items included in such costs, and the extent of the work involved, and hence are not reliable for comparative purposes. On account of extremely variable local conditions, such as the length of transmission lines, height of dam, provision for extensions, and costs of land, construction materials, labor, the cost varies widely in different localities. The author's experience with water powers ranges from $80 to $200 per kilowatt of installed capacity not including local distribution. The maximum cost for a profitable investment varies with the nature of the market and the value of the output. High-load-factor markets, in districts where fuel is expensive, are capable of supporting much more costly developments than other localities where less favorable conditions obtain.

44. Value of the Output.—The value of the output is usually fixed by competition with other possible sources of power. Steam power is the principal competitor of water power, although in districts remote from the coal fields, oil combustion engines are extensively used. Modern steam plants of extremely large capacity, close to coal mines, can produce power at a very low cost. (See Sec. 476.) In such instances, competition by water power is difficult. For smaller steam plants and plants remote from the mines the cost of power rapidly increases.

45. Auxiliary Power Plants.—To furnish supplementary or auxiliary power to the proposed water-power system, a steam plant or other water powers may be used. Oil and gas engines are used infrequently. Except for very large power systems, the arrangement usually consists of one or more water powers supplemented by a single steam plant of capacity sufficient only to supplement the water powers during periods of insufficient stream flow. Low-head developments, on large streams without storage, and high-head, storage developments supplement each other admirably, particularly for primary-power loads as explained in Sec. 56.

It is seldom that auxiliary plants are constructed solely as a provision against breakdown of transmission lines or water-power apparatus and conduits. Interruptions of service, other than those due to lack of water during droughts or loss of head during floods,[3] are very infrequent in properly designed and constructed modern plants. For important markets, spare hydro-electric units and duplicate transmission lines reduce the chances for interrupted service, making them comparable with those existing in steam plants, particularly if more than one water power is connected up to the system.

The interconnection of neighboring power systems for the purpose of an interchange of power has been receiving considerable attention during recent years.[4] As explained in Sec. 40, the resulting increase in market almost invariably effects an increase in load factor. Moreover, the minimum flow of the different streams, if they are somewhat remote, may not occur simultane-

[3] See Sec. 49.
[4] See Reference 1, Sec. 46.

ously. Actual interconnections have shown materially beneficial results, not only in the improvement of market requirements and water-power output, but in the matter of additional safeguards against interruptions and more uniform auxiliary steam plant output.

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CHAPTER VII

OUTPUT CAPACITY AND FLOW DEMAND

BY WILLIAM P. CREAGER

47. General Considerations.—The definitions of output capacity and flow demand, together with a description of their functions in calculations of power available, is given in Sec. 39. As indicated in Fig. 33, they depend upon the market requirements, the installation capacity, the efficiency, the pondage, and the head. Market requirements and installation capacity are treated in Chapter VI.

48. Efficiency of Apparatus.—The unavoidable losses in the generation and transformation of the energy of a water-power development, and in its transmission to the point of ultimate use, frequently amount to a large part of the total energy available. As will be seen in Sec. 49, the losses in the conduit system are accounted for in the use of net or productive head at the turbines and are not charged against the efficiency of the station. The energy lost in the various types of apparatus required for the generation of mechanical energy by the turbine, its conversion to electrical energy by the generator, its transformation to high voltage suitable for transmission, its transmission and its transformation to low voltage suitable for local distribution, is measured by the efficiency of the apparatus.

Efficiency is the ratio between the energy delivered by a machine or other form of apparatus and the energy supplied to it, usually expressed as a percentage. Thus, if 8000 kw. are being supplied to a turbine by the water passing through it and 6400 kw. are delivered to the generator shaft, the efficiency of the turbine is 80 per cent. If the generator then delivers 5950 kw. to the busbars, the efficiency of the generator is 93 per cent. The combined efficciency of several machines is the product of their individual efficiencies. In the preceding case, the combined efficiency of the turbine and generator is 74.4 per cent. The efficiency of the station is the ratio between the energy delivered to a given point and the energy supplied to the turbine, and is the product of the efficiency of all the parts of the system.

The efficiency of the various forms of apparatus used in a hydro-electric system is not constant, but changes with the amount of power handled. The variation of efficiency with power depends upon the characteristics of the apparatus and is often determinable only by tests. Manufacturers' guaranteed efficiencies of apparatus and computed transmission-line efficiencies for a typical, medium-size, single-unit hydro-electric station are given in Fig. 39 and for a three-unit plant in Fig. 40. It will be noted that, with the excep-

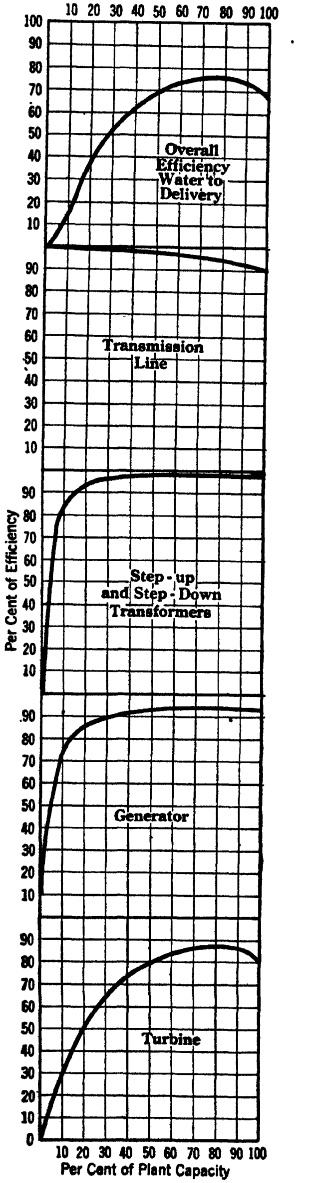

FIG. 39.—Efficiency of Typical Single Unit Hydro-Electric Development.

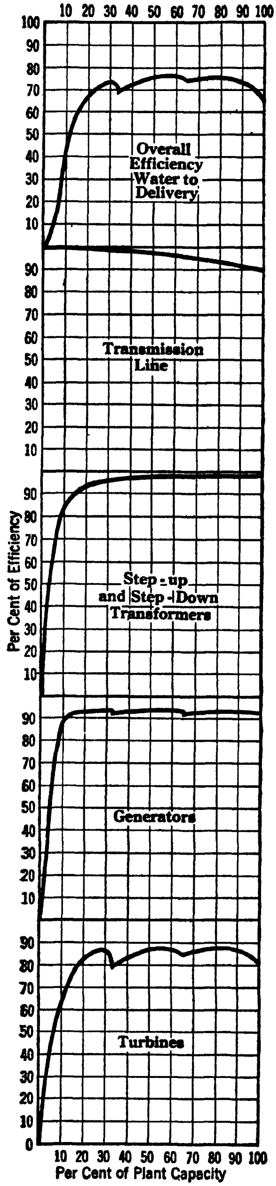

FIG. 40.—Efficiency of Typical Three Unit Hydro-Electric Development.

tion of the transmission line, the efficiency for small loads is quite low and, principally for this reason, two or more units are usually provided.

Figure 41 shows a typical comparison of over-all efficiencies between plants containing the following combinations of similar units:

(*a*) A single unit;
(*b*) Two units of equal size;
(*c*) Three units of equal size;
(*d*) Two units, one of one-third and the other two-thirds of station capacity;
(*e*) An infinite number of units.

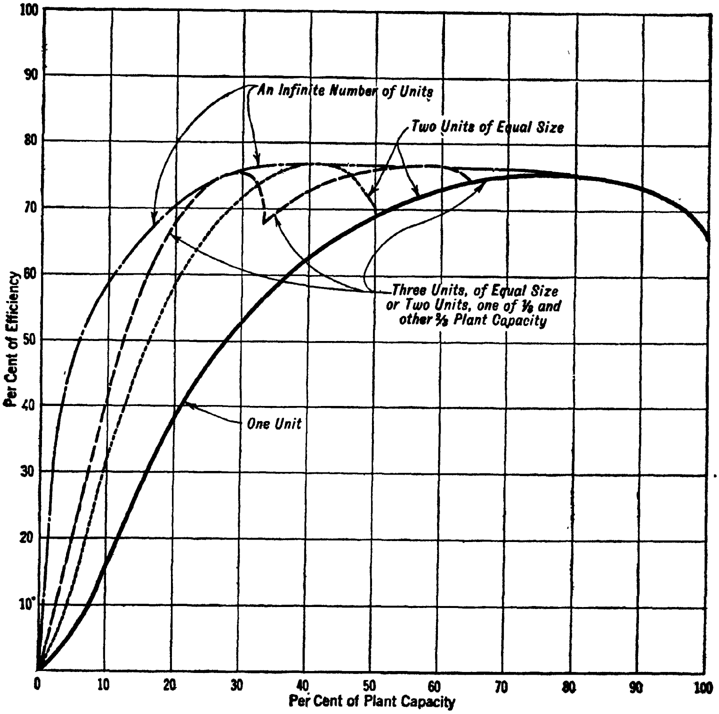

FIG. 41.—Typical Comparison of Overall Efficiency. Plants of Different Numbers of Similar Units.

The scheme providing two units of unequal size (*d*) is an unusual type advocated by very few engineers. It has the same efficiency as a three-unit plant, but is probably not much, if any, less costly than a three-unit plant and has the objection of non-interchangeability of parts.

It is seen in Fig. 41 that a two-unit plant has decidedly better average efficiency than a single-unit plant, but the advantage of a three-unit plant over a two-unit plant is less marked. It is interesting to note the rather small

difference in efficiency, from 30 to 100 per cent of station capacity, between a three-unit plant and a theoretical plant containing an infinite number of units.

Actual efficiencies of apparatus are frequently found to be several per cent higher than guaranteed, but it is not safe to count on this possiblity. The efficiency of turbines, generators, and other types of apparatus varies somewhat with their size. Small units may have slightly lower, and large units higher, efficiencies than those shown in the foregoing curves. The maximum efficiencies so far obtained for very large turbines and generators are about 94 and 97 per cent, respectively, at points of best load. Transmission-line losses vary with the size of wire used, and therefore the efficiency can be increased to any economical limit by simply increasing the size of the wires. Transmission-line efficiency is seldom less than 90 per cent at full load. The losses vary as the square of the power transmitted.

As the load, and hence the efficiency, is not constant, the average station efficiency for use in computing energy available from stream flow depends upon the shape of the load curve, the apparatus employed, and the variation in head. For units over 1000 kw. capacity, having a variation in head not in excess of about 10 per cent, the following are conservative values of average efficiency for a plant having two or more units generating for a typical nearly general city load on a large system.

Much higher efficiencies can be obtained for extremely large units.

	Per Cent Efficiency	Per Cent Accumulative Efficiency
Turbines	83	83.0
Generators (including excitation)	93	77.2
Step-up transformers	98	75.6
Transmission line	95	71.8
Step-down transformers	98	70.3
Combined or station efficiency	....	70.3

The efficiency of turbines decreases with age if proper maintenance is neglected, and is very rapid if silt under high heads, or acid, is present in the water. The efficiencies of generators and transformers are practically permanent if the insulation is not allowed to deteriorate. Transmission-line efficiency changes only by reason of damage to the wire or insulators or by short-circuiting. The efficiencies given in the foregoing tabulation are for new plants or for plants only a few years old and properly maintained. It is impossible to predict possible efficiencies after a number of years have passed; but under ordinary conditions little, if any, allowance need be made for reductions in efficiency, if there is ample opportunity for thorough inspection at frequent intervals and for proper maintenance.

The variation of turbine efficiency with changes in head is given in Sec. 318.

49. Head.—The *gross head* is the total fall or difference between the elevation of water surface in the diversion pond and that in the lower end of the tail

race. The gross head varies from time to time with the elevations of these water surfaces, which may change on account of draft from the pond or change in elevation of tail-water with discharge of the river. The gross head may increase or decrease during floods, depending upon whether head-water or tail-water rises faster. Usually the head is reduced during floods because the discharge over the dam is frequently less restricted than that in the natural stream channel.

The *net head* [1] is the gross head less all losses in the conduits and tail race. Losses within the turbine casing, the turbine, and the draft tube are not included in the conduit losses, being chargeable against turbine efficiency. The net head is therefore the effective head at the turbine available for the production of power. The net or effective head for encased reaction turbines has been defined by the Testing Code of the Machinery Builders' Society [2] as the difference between the elevation corresponding to the pressure in the penstock near the entrance to the turbine casing and the elevation of tail-water, the above distance being corrected by adding the velocity head in the penstock at the point of measurement and subtracting the residual velocity at the end of the draft tube.[3] For turbines in open flumes, the net head is defined as the difference between the elevation of water surface above the turbine and the elevation of tail-water, less the residual velocity at the end of the draft tube.

For impulse wheels, the net head is the difference between the elevation corresponding to the pressure in the penstock near the entrance to the turbine nozzle casing and the elevation of the jet of water where it becomes tangent to the bucket circle, the above distance being corrected by adding the velocity head in the penstock at the point of measurement.[4]

The net head varies with the load on the unit because the friction and velocity losses increase approximately as the square of the discharge. The net head at full load for low-head developments with very short or no conduits or tail race, is practically equal to the gross head; but for high-head developments, requiring long conduits, the net head may be only 90 per cent, or even less, of the gross head.

Drawdown for pondage varies with the topographical features at the site and the nature of the load. For low-head developments, the average drawdown required is not often in excess of 5 per cent of the total head, and for very high-head developments it is negligible.

If the lake at the diversion dam is to be used for storage, the extent of drawdown may be a large percentage of the total head, depending upon the relative value of water and head. It is evident that the last of the water from the reservoir is of no practical value if all of the available head is used up to provide it. Special cases of excessive drawdown are those in which the turbines are installed at storage reservoirs as supplementary units to the main development for which the reservoir is regulated.

[1] See also Sec. 76.
[2] Approved, Oct. 11, 1917.
[3] See Eq. (56) of Sec. 76.
[4] See Eq. (58) of Sec. 76.

The head may be materially reduced or practically eliminated during periods of excessive floods at low-head developments. This has an important bearing on the choice of station capacity for primary power if auxiliary plants are not provided.

The *productive head* is that head which, when combined with average discharge, gives average output. The net head corresponding to average discharge is greater than the productive head and, if used, will give too large a calculated output, as explained in Sec. 50.

50. Energy, Work and Power.—The unit of *energy* is the foot-pound, or one pound raised a distance of one foot. *Work* is the utilization of energy and is measured by the same unit. The potential energy of a volume of water is the product of its weight times the head, or vertical distance it may be lowered. In the transformation of this energy into useful work, part will be lost in the conduit system and part in the apparatus of the development.

If a volume of water in a storage reservoir is allowed to pass through the turbines under a constant net head, the energy of this water delivered in the form of work is:

$$K = wV(H - h_f)e, \qquad (7)$$

where K = energy in foot-pounds;
w = the weight of 1 cu. ft. of water in pounds;
V = the volume of water, in cubic feet;
H = the gross head, in feet;
h_f = the head lost in the conduit system and tail race;
e = the station efficiency, expressed as a fraction.

Also let q = the discharge of a stream in cubic feet per second;
t = a period of time in seconds;
T = a period of time in hours.

If the discharge is passed through the turbines, the total volume, v, used during the period, t, is:

$$V = qt,$$

which, if used in Eq. (7), gives:

$$K = wqt(H - h_f)e, \qquad (8)$$

which is the delivered energy of the stream during the period, t.

Power is the rate of work, or the work done in a specified time. The usual units of power are the horse power (h. p.) and the kilowatt (kw.) these being, respectively, 550 and 737 ft.-lb. per second. The power, p, corresponding to K ft.-lb. of energy in time, t, is:

$$p_H = \frac{K}{550t}, \text{ horse power}, \qquad (9)$$

$$p_K = \frac{K}{737t}, \text{ kilowatts}. \qquad (10)$$

Substituting the value of K from Eq. (8), and using the usual value [5] of 62.5 lb. in place of w, we have for power:

$$p_H = \frac{q(H - h_f)e}{8.8}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (11)$$

$$p_K = \frac{q(H - h_f)e}{11.8}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

Energy may also be expressed in horse power-hours (h.p.-hr.) or kilowatt-hours (kw.-hr.) and Eqs. (11) and (12) may be converted into:

$$\text{h.p.-hr.} = p_H T = \frac{q(H - h_f)eT}{8.8}, \quad . \quad . \quad . \quad . \quad . \quad . \quad (13)$$

$$\text{kw.-hr.} = p_K T = \frac{q(H - h_f)eT}{11.8}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

where T is the period of flow in hours.

Equations (11) to (14) inclusive, are the basic expressions used in calculations for power and energy. Equations (11) and (12) are correct for power at any instant, and Eqs. (13) and (14) are correct only if q is constant during the period, T. If q varies during the period, h_f also varies, but not in direct proportion because it is approximately proportional to the square of the conduit velocities. Therefore, as indicated more clearly below, the net head, $H - h_f$, corresponding to average discharge, cannot be used in Eqs. (11) and (12) to calculate average power nor in Eqs. (13) and (14) to calculate energy output during a given period, unless the friction head, h_f, is very small or q is practically constant.

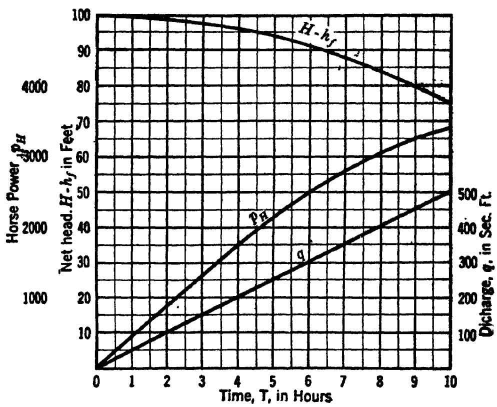

FIG. 42.

In Fig. 42, values of q, $H - h_f$, and p_H are indicated for a case where q varies from zero to 500 sec.-ft. at the end of a ten-hour period. The equations are,

$$H - h_f = 100 - \frac{q^2}{10{,}000},$$

$$p_H = \frac{q(H - h_f)e}{8.8} = \frac{q(H - h_f)}{11},$$

efficiency, e, being assumed equal to 80 per cent.

[5] The weight of one cubic foot of water varies, for all practical purposes, between very narrow limits, and for ordinary computations may be taken as 62.5 lb. per cubic foot.

The average power during the period, calculated from the curve, is 1990 h.p. The average discharge during the period is 250 sec.-ft. The net head, corresponding to average discharge, is seen to be 93.75 ft. If this value of net head were used to determine the average horse power during the period, the erroneous result would be,

$$p_H = \frac{250 \times 93.75}{11} = 2130 \text{ h.p.},$$

which is seen to be considerably greater than the true value of 1990.

Calculations necessary to determine the true average output, as indicated for Fig. 42, are too laborious for practical use, and an approximate correction is made by multiplying the head corresponding to average discharge by a coefficient, C, or, letting

P_H = average horse power during a given period;
P_K = average kilowatts during the period;
Q = average discharge during the period;
H_f = conduit losses corresponding to Q.

Then

$$P_H = \frac{Q(H - CH_f)e}{8.8} = \frac{Qhe}{8.8}, \qquad (15)$$

$$P_K = \frac{Q(H - CH_f)e}{11.8} = \frac{Qhe}{11.8}, \qquad (16)$$

$$\text{h.p.-hr.} = \frac{Q(H - CH_f)eT}{8.8} = \frac{QheT}{8.8}, \qquad (17)$$

$$\text{kw.-hr.} = \frac{Q(H - CH_f)eT}{11.8} = \frac{QheT}{11.8}. \qquad (18)$$

$(H - CH_f) = h$ is termed the *productive head*. It is always equal to or smaller than the net head corresponding to Q. In the preceding example, $C = 2$ and the productive head is $100 - 2 \times 6.25 = 87.5$ ft. as compared with the head, 93.75 ft., corresponding to average Q.

The value C is theoretically obtained by dividing the total period into a number of equal intervals of time, so small that the discharge is practically constant for each interval.

Let q_1, q_2, etc. = the average discharge during each interval;
n = the number of intervals.

Then

$$C = \frac{q_1^3 + q_2^3 + \cdots q_n^3}{nQ^3} \qquad (19)$$

In practice, however, the load curve or variation in demanded power output is first known and Q is desired. In such cases, C may be approximately determined by substituting power in place of discharge in Eq. (19),

C tends to increase as the load factor decreases. An approximate relation between C and load factor is indicated in Fig. 43 in which the plotted points were computed from a number of typical load curves of public utility plants. The error involved in the use of Eqs. (11) to (14) inclusive is not on the side of conservatism, but will ordinarily be negligible if the conduit losses are small, and will not exceed about 4 per cent unless the head lost in the conduit system at full load is in excess of about 10 per cent and the load factor less than 30 per cent. Therefore, the use of Fig. 43 to determine the coefficient, C, for use in the more accurate Eqs. (15) to (18), inclusive, will ordinarily give results well within the degree of precision desired.

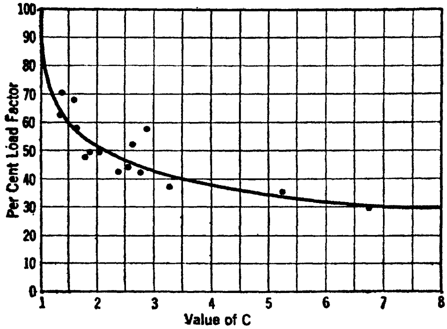

FIG. 43.—Values of C in Equation 19.

51. Output Capacity.—The water-power output capacity depends upon the extent to which the installation may be utilized within the limits of the market requirements. The various factors affecting output capacity are shown in Fig. 33 and described briefly in Sec. 39. The following examples will serve to indicate the various limitations of a water-power development as affected by market requirements, capacity, pondage, stream flow, and auxiliary power plants.

Case I.—Water-power capacity equal to peak demand.
  (*a*) With adequate pondage;
  (*b*) Without adequate pondage.
Case II.—Water-power capacity less than peak demand.
  (*a*) With adequate pondage;
  (*b*) Without adequate pondage.

Case Ia.—The load curve of Fig. 34 is used in Fig. 44 to explain the distribution of loads for this case with adequate pondage. In Sketch *A* of Fig. 44 the uniform stream flow, reduced to kilowatts by Eq. (16) of Sec. 50, is shown slightly greater than that corresponding to average power demand.

Assuming the pond to be full at 8.15 A.M., a portion, P, of the load between 8.15 A.M. and 10.40 P.M. is generated from water drawn from pondage. Between 10.40 P.M. and 3.30 A.M., the excess flow, F (equal to P) not required for power, is used to refill the pond, and the remainder, L, of the excess flow must be wasted over the dam.

In Sketch B, the flow is shown to be exactly equal to that required for average demand. The pond is drawn down between 7.40 A.M. and 11.00 P.M. and refilled during the remainder of the day. As the flow corresponds to average demand, no water is wasted and the required draft from the pond is

greater than for any other flow, for this case corresponding to an output of 25,200 kw.-hr.

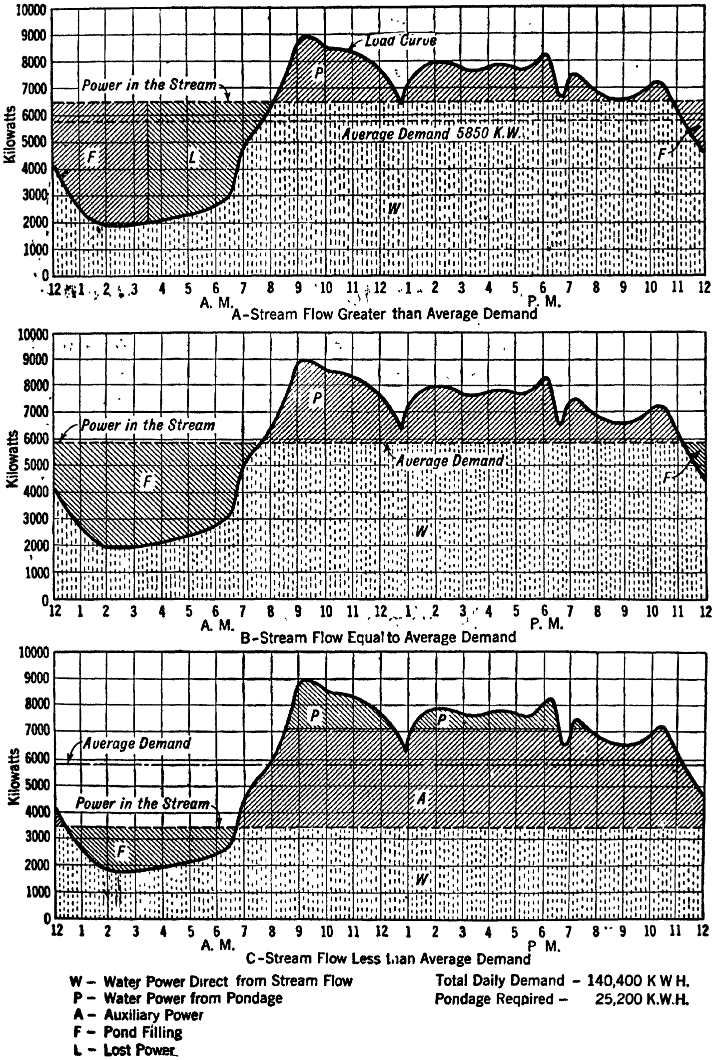

FIG. 44.—Typical Distribution of Load, Adequate Installation and Pondage.

In Sketch *C*, the flow is less than required for average demand; consequently a deficiency of water-power output exists and the auxiliaries are called upon to supplement the water power. The early morning excess flow,

F, is retained in the pond and is used for the peak load, P. The auxiliaries supply the block of power, A. For economy of operation of the auxiliaries, the output of the water power is adjusted to require uniform supplementary power in so far as the load demand will permit.

It will be noted that, with adequate pondage, no deficiency of water-power output occurs except when the stream flow is less than that corresponding to average demand, and that the average deficiency is equal to the difference between the power in the stream and the average demand.

The relation between the energy in the stream and the energy output for Case Ia is indicated by Curve ABC of Fig. 46, expressed in both kilowatts and stream flow by Eq. (16) of Sec. 50. The curve is based on a productive head, h, of 33 ft. and average station efficiency of 70 per cent to point of delivery. This curve is the *output-capacity* relation for Case Ia, and shows a simple relation which may be expressed as follows: For adequate pondage and installation, the output of the water power is equal to the power in the stream at all times during which the power in the stream is equal to or less than the average demand, and the output is equal to the average demand for all greater flows.

Case Ib.—If no pondage is available, the flow cannot be regulated, and all demanded load above the "power in the stream" line of Sketches A, B, and C, of Fig. 44, must be supplied by the auxiliaries. In other words, the auxiliaries must supply not only the power, A, indicated for Case Ia, but the power, P, which, in the previous case, was taken from pondage, the discharge F, which with pondage could be stored and used later in the day, being wasted.

Curve $AFIGC$ of Fig. 46, plotted from Fig. 44, shows the output-capacity relation for Case Ib. When the power in the stream is equal to or greater than the peak demand of 9000 kw., ample flow for all purposes is available and the water-power output is equal to the average demand of 5850 kw., as indicated between points G and C; when the power in the stream is equal to or less than the minimum demand of 1800 kw., no pondage is required, no water is wasted, and the output is equal to the power in the stream as indicated between points A and F.

Between points A and F and between G and C, the curve coincides with that for Case Ia; but between points F and G a considerable amount of flow which, with pondage, would be utilizable, is wasted, and the curve for Case Ib falls below that for Case Ia by the amount of such waste.

If a portion of the required pondage is available, the auxiliaries will be required to supply a portion of the power, P, in addition to power, A, Fig. 44. The output-capacity relation for this case is shown by curve $ADEC$ of Fig. 46, which is drawn for a pondage of 15,700 kw.-hr.

Case IIa.—As indicated in Fig. 45, the installation is inadequate to the peak demand. In Sketch A the power in the stream is shown to be in excess of installation. No pondage is required and all flows, L, above the load curve and the installation capacity are wasted. The auxiliaries would be called upon for the load, A.

If the power in the stream is less than plant capacity as indicated in Sketch B, the excess early morning flow, F, is ponded and utilized between 6.15 A.M. and 12.45 A.M., the whole flow being utilized for power. For this

distribution of the load, the auxiliary plants would be required to provide the deficiency, *A*, of output. A better distribution of loading, however, is pos-

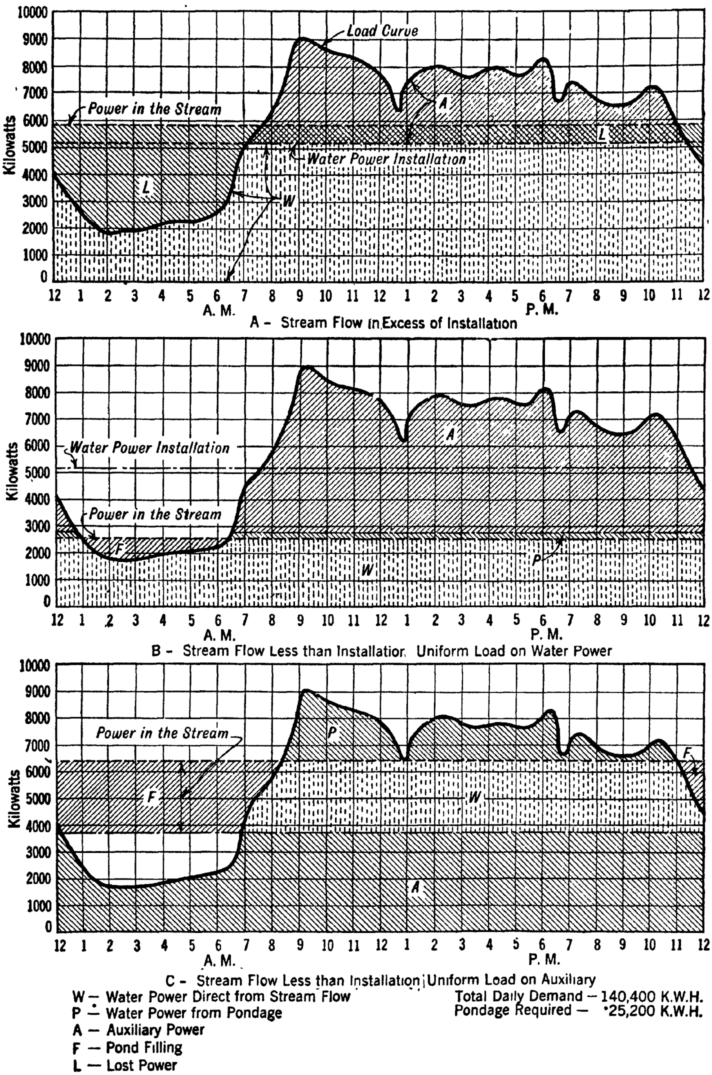

FIG. 45.—Typical Distribution of Load, Inadequate Installation and Adequate Pondage.

sible. Auxiliaries operate with best efficiency on a uniform load. Therefore, with adequate pondage, the distribution of load shown in Sketch *B* is less

advantageous than that indicated in Sketch *C* where the flow is utilized to allow the auxiliaries to operate on a more uniform output. The stream flow, *F*, during the early morning, is held in the pond and utilized during the day to carry the peak of the load, *P*. This distribution requires considerably more pondage than that shown in Sketch *B*. In both cases the whole flow is utilized.

From Fig. 45, it can be calculated that the average demand below the water-power installation is 4250 kw. Therefore the line *HJ* of Fig. 46 indicates, for this example the maximum possible output from water power with ample stream flow, and the line *AHJ* for all flows with ample pondage.

Case IIb.—Without pondage, the water-power output capacity, for flows less than 4250 kw. is exactly the same as for Case I*b*. Therefore the line *AFIJ* of Fig. 46 indicates the output for all flows with no pondage.

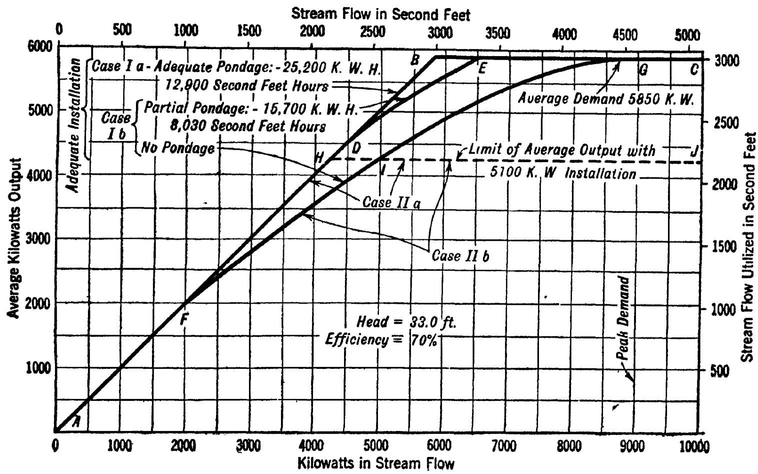

Fig. 46.

A daily load curve has been used in the foregoing discussion; but a similar study, based on a weekly load curve as indicated in Fig. 36, must be used if the Saturday and Sunday loads are different from those of week days. Moreover, studies are usually made for the four seasons of the year if the seasonal load is variable.

The use of the output-capacity relation, indicated in Fig. 46, for calculations of power available from stream flow, is explained in Chapter VIII. With ample pondage, Cases I*a* and II*a*, the relation is so simple that an actual diagram need not be used; but, when pondage is inadequate, a diagram is very convenient for reducing stream flow to usable flow or power output.

There are many special cases, involving conditions affecting materially the output capacity, and flow demand, which cannot be described here but will be obvious to the engineer. The special case of variable head due to excessive draft on the pond or high tail-water during floods is the most com-

mon. The flow demand for plants requiring excessive reduction of head for pondage or storage will vary according to the drawdown of the reservoir. In some cases of low-head developments, tail-water during floods rises so rapidly that the head is practically eliminated during such periods, and only a small amount of flow under a materially reduced head can be utilized. Obviously, this deficiency cannot be supplied except by auxiliary plants.

52. Flow Demand.—The flow demand, or stream flow required to meet all power demands, for the foregoing examples, may be obtained directly from Fig. 46. This has been done and the result tabulated in Table X.

For Case I*a*, with adequate pondage, the flow demand is equal to the average output demand of 5850 kw.

For Case I*b*, with inadequate pondage of only 8030 sec.-ft. hr., the flow must be such that the demanded load, *P*, above the power in the stream equals the available pondage, as indicated in Sketch *A* of Fig. 44, where, if *P* equals the available pondage of 8030 sec.-ft. hr., the flow demand corresponds to 6500 kw. or 3330 sec.-ft.

For Case I*b*, with no pondage the flow must correspond to the peak demand of 9000 kw. as no regulation is possible.

For Case II*a*, the flow must correspond to the average of the load demand below the water-power installation.

For Case II*b*, the flow must correspond to the capacity of the installation.

TABLE X

TABULATION OF FLOW DEMANDS FOR PRECEDING EXAMPLES

Case	Figure	Installation	Pondage	Flow Demand	
				Kw.	Sec.-ft.
I*a*	43	Adequate	Adequate	5850	3000
I*b*	43	Adequate	8030 sec.-ft. hr.	6500	3330
I*b*	43	Adequate	None	9000	4620
II*a*	44	5100 kw.	Adequate	4250	2180
II*b*	44	5100 kw.	None	5100	2610

The flow demand is the ultimate objective of storage, the purpose of which is to make the flow at all times sufficient for maximum possible use of the installation. It is obvious that the flow demand may not be constant, particularly if storage is provided at the development and the head is considerably reduced in order to draw from the reservoir. High tail-water during floods often reduces materially the available head at low-head plants and increases the flow demand.

CHAPTER VIII

STORAGE AND POWER AVAILABLE

By William P. Creager

53. General.—In computations for power available from a hydro-electric development during a given period, it is customary to convert the number of kilowatts of power needed to meet market requirements to its equivalent in second-feet of flow-demand, as explained in Sec. 52. The flow demand is then used as a basis for estimating storage requirements or, in case of inadequate storage or pondage, is used jointly with the output-capacity curves of Sec. 51 for computing power available from the stream. Equations for the conversion of kilowatts to second-feet are given in Sec. 50.

The application of hydrographs, mass curves, duration curves, and analytical methods to calculations of storage requirements and estimates of output will be given in this chapter. It will be seen that each of these methods has particular usefulness in special cases; but frequently the choice of method, while depending to a considerable extent upon the nature of the problem, is dictated by the preference of the engineer and the manner in which the results of his calculations are to be published.

Methods suitable to four general cases are outlined briefly below. Where several methods are indicated, the first is usually the author's preference. Pondage in excess of an amount sufficient to control the flow, and make it conform to the varying daily and weekly demand, is classed as storage.

Case A.—No storage—adequate pondage.
- Analytical methods.
- The hydrograph.
- The duration curve.

Case B.—No storage—no pondage.
- The duration curve.

Case C.—Storage at the plant—adequate pondage.
- Analytical methods.
- The mass curve.
- The hydrograph.

Case D.—Remote storage—adequate pondage.
- Analytical methods.

Case E.—Remote storage—no pondage.
- A combination of the analytical method and the duration curve.

An estimate of the minimum daily output of the water-power development is essential to the determination of capacity of auxiliaries to provide continuous

output. An estimate of the output in a year of minimum flow is usually essential to indicate the worst possible yearly operating revenue. The average yearly output is used to compute the average return on the investment.

54. Storage.—In most cases of highly developed streams, the natural flow during a large part of the year is deficient for the flow demand, and regulation by storage is desirable. If it is not feasible to obtain reservoirs of sufficient capacity to effect complete regulation, partial regulation is often found advantageous.

The extent to which streams can be economically regulated is affected by many practical considerations and depends upon the relation between the value of the benefit from storage and the annual operation, interest, and other overhead costs of the reservoirs and dams. The benefit derived from storage is measured by the resulting decrease in energy required from the auxiliaries to maintain continuous output and by the reduction in peak demand of auxiliaries; or, in the case of an independent water power by the resulting increase in both the energy output and the value of such output, or by the extent to which the water power is made continuous and reliable.

The capacities of storage reservoirs are often affected greatly by the gradual deposition of silt. In some districts these deposits may be enormous and may, in the course of a few years, completely fill the reservoir and destroy its usefulness. Sluices in the dam are never effective in removing sediment, except that which has been deposited near the dam. Silting from forest-covered areas is negligible. In regions subject to violent rainstorms and not protected by vegetation, as in the Southwest, streams are heavily laden with silt.

Many tests of the silt content of streams have been made by the U. S. Reclamation Service and others. Records of sixteen years of observations of the Rio Grande show an average of $1\frac{2}{3}$ per cent of silt carried in suspension, and several monthly averages in excess of 10 per cent. Unfortunately, no satisfactory method has yet been devised for measuring the amount of sand, gravel, and even large boulders which are projected along the bottom. It is claimed that the percentage of matter carried in this manner is relatively small in most cases. The nature of deposits against existing dams and natural obstructions serves to indicate the nature of the transported materials.

To measure the percentage of silt carried in suspension, a given volume of water is taken and evaporated, and the residue weighed. The dry weight of 1 cu. ft. of deposited silt varies with the nature of the material, and has been found to range between 50 and 90 lb. However, samples can be taken from natural deposits and the weight accurately determined.

A check can be made by measuring the depth of silt in existing reservoirs on similar streams, if the original bottom elevation is known.

Proper allowance should always be made for estimated reduction in capacity from silting, either by providing excess capacity to retain the silt for a long period, or by control of other storage sites for future use.

55. Tabulating Stream Flow.—The following tables indicate the flow of a typical river, the actual discharges of which have been altered to cover desired

conditions and to clarify explanations and examples. Tables XI and XII are in the form in which such data are usually presented in the U. S. Geological Survey Water Supply Papers. Table XIII is a compilation of the average monthly flows for the years of record, and is in the form frequently used in engineering reports.

Tabulations are sometimes more conveniently made in second-feet per square mile of drainage area.

The " Typical River " will be used in all the examples that follow.

TABLE XI

DAILY DISCHARGE, IN SECOND-FEET, OF TYPICAL RIVER FOR THE YEAR ENDING DEC. 31, 1918 (MINIMUM YEAR OF RECORD)

Drainage Area, 2000 Square Miles

Day	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	3200	3,850	10,600	6,000	8700	1100	650	810	4500	3560	1710	3015
2	3350	5,500	10,200	6,400	8500	1200	670	1040	5100	4500	1830	3500
3	3500	12,400	10,300	6,000	7100	600	1050	1660	5500	5100	2070	3700
4	3700	10,200	9,500	5,300	6800	900	1000	1970	5700	4900	2380	3900
5	3860	8,500	10,700	5,400	7000	1400	1210	1610	5600	6000	2490	3800
6	3650	5,300	11,700	5,700	6900	1600	1200	1600	5700	7000	2370	3550
7	3500	4,900	12,000	5,200	6400	700	1340	1980	5650	6100	2480	3770
8	3400	4,900	11,900	5,300	6200	800	1590	2250	6000	5900	2340	3880
9	3450	4,800	12,000	5,000	5800	750	960	2210	6400	6000	2410	4000
10	3650	4,850	12,600	5,200	5900	850	1010	1920	5000	4400	2500	3850
11	3640	4,500	12,400	6,000	3400	700	1130	2000	5100	4500	2470	4050
12	3510	4,550	12,500	7,200	4000	680	1200	1880	4800	4100	2380	4480
13	3500	4,300	12,350	8,300	4500	650	1210	1470	5300	3420	2200	4500
14	3540	4,220	12,600	8,200	3800	620	1100	1520	5600	3180	2110	4630
15	3710	4,230	12,800	8,250	3000	600	1080	1570	4260	2500	1990	4850
16	4000	4,400	12,300	8,250	2500	550	960	2040	3000	2180	2020	4300
17	4240	4,700	12,000	10,000	2600	650	900	2460	2770	2100	2170	4150
18	4160	4,900	12,200	9,800	2400	710	780	2710	2850	2220	2250	4200
19	3920	5,200	10,800	10,300	2600	920	780	2500	2970	2470	2160	4000
20	3950	5,600	12,300	10,200	2700	670	1040	2660	2860	2610	2260	3580
21	4100	7,000	11,700	9,700	2200	620	960	2840	2870	2150	2300	3570
22	4250	8,500	11,750	9,300	2300	640	970	3010	2880	2270	2370	3520
23	4320	9,600	11,500	8,500	2400	700	920	3270	2900	2090	2390	3480
24	4250	8,700	10,500	8,400	2000	850	1000	3380	2890	2010	2390	3500
25	4200	9,400	8,400	8,500	1600	1030	1150	3540	2500	1400	2390	3610
26	4150	10,000	5,900	9,500	1700	1310	1370	3750	2260	1570	2380	3870
27	4080	10,500	6,000	8,800	1600	1600	1280	3800	2490	1480	2740	4000
28	3950	11,000	6,200	9,000	1500	1000	1140	3900	2370	1600	2710	3850
29	3960	....	6,400	8,900	1200	780	980	4000	2000	1580	2780	3760
30	3850	....	6,550	8,800	1000	660	860	4100	2850	1570	2940	3700
31	3800	....	6,700	....	1100	.....	800	4200	.....	1550	.....	3570

56. The Hydrograph.—A hydrograph is a graphical indication of the rate of flow of a stream during a given period, the discharge being plotted as ordinates and time as abscissae. Figs. 47 and 48 are, respectively, typical hydrographs of a very flashy stream and of a stream having unusually steady flow. Fig. 49 is a minimum year daily hydrograph of Typical River, plotted from Table XI, and Fig. 50 is a monthly hydrograph of the same year, plotted from Table XII.

TABLE XII

MONTHLY DISCHARGE OF TYPICAL RIVER FOR THE YEAR ENDING DEC. 31, 1918

Drainage Area, 2000 Square Miles

Month	Discharge in Second-feet				Runoff		Accuracy
	Maximum	Minimum	Mean	Mean per Square Mile	Depth in Inches on Drainage Area	Total in Acre-feet	
January	4,320	3200	3,820	1.91	2.20	235,000	B
February	12,400	3850	6,650	3.32	3.46	370,000	B
March	12,800	5900	10,500	5.25	6.06	647,000	B
April	10,300	5000	7,710	3.85	4.30	459,000	A
May	8,700	1000	3,850	1.93	2.22	237,000	A
June	1,600	600	860	0.43	0.48	51,200	A
July	1,370	650	1,040	0.52	0.60	64,100	A
August	4,200	1040	2,500	1.25	1.44	154,000	A
September	6,400	2000	4,020	2.01	2.24	239,000	A
October	7,000	1400	3,300	1.65	1.90	203,000	A
November	2,940	1710	2,330	1.16	1.30	139,000	A
December	4,850	3015	3,880	1.94	2.24	239,000	B
The year	12,800	600	4,205	2.10	2.37	253,100	

TABLE XIII

AVERAGE MONTHLY DISCHARGE OF TYPICAL RIVER FOR THE YEARS 1900 TO 1919, INCLUSIVE

Drainage Area, 2000 Square Miles

Year	Jan.	Feb.	March	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1900	4000	6500	13,100	14,100	9,100	1210	910	2250	3700	4000	2500	4000
1901	7400	6400	18,700	19,700	5,000	1380	1360	2400	3900	4700	2900	4300
1902	4300	7000	14,500	18,800	4,200	3050	1420	3200	4600	6500	5900	6500
1903	7200	3900	10,400	17,900	8,800	4300	3500	3400	6500	6800	6500	7700
1904	8500	4500	16,200	15,200	3,100	1190	3330	5000	4700	5000	6900	7400
1905	8600	7200	24,900	18,000	7,200	3020	1020	2450	4100	3600	2300	4000
1906	7600	9200	22,000	10,200	7,900	2500	1580	2500	4000	7100	3300	7400
1907	8400	6500	15,300	16,300	6,200	970	2710	3000	4500	7400	5500	4900
1908	6800	8700	11,100	8,800	2,700	1070	2400	3500	4800	5300	7200	4700
1909	6500	6300	16,200	17,900	5,600	1430	990	2050	5000	3500	2400	5000
1910	4600	6300	8,800	14,400	6,300	1510	1630	2600	6000	3800	7500	8200
1911	8200	7300	10,400	13,400	3,800	2130	1750	4500	4800	7700	3700	6700
1912	6200	4300	12,600	16,100	4,300	1040	1460	3700	4400	5600	7900	3500
1913	8000	7900	18,300	19,500	6,500	920	1740	2900	4100	4100	5200	5200
1914	5900	3700	14,500	12,700	5,400	3500	1370	2700	4900	7900	4100	5600
1915	4900	6200	20,700	14,300	10,700	850	1240	2100	3600	5000	2500	4500
1916	7800	7500	16,600	20,600	7,400	1630	1480	3900	4900	5900	8400	3000
1917	5500	8100	9,800	13,500	8,500	2750	1670	4100	4200	4400	4500	5600
1918	3820	6650	10,500	7,710	3,850	860	1040	2500	4020	3300	2330	3880
1919	5200	6000	12,100	10,600	9,600	1860	1550	2800	4300	6200	4900	5900

In Fig. 49 a flow demand of 3000 sec.-ft., or 36,000 sec.-ft. months per year has been indicated. This demand corresponds to Case I*a* as shown in Table X of Sec. 52. If all flows above the flow demand of 3000 sec.-ft. are discarded, the total area below the demand line and the hydrograph, or 29,200 sec.-ft. months, is the total natural flow available for power or 21,300,000 sec.-ft. hr. during that year. From Eq. (18) of Sec. 50, this corresponds to 41,700,000 kw.-hr. under a productive head of 33 ft. and 70 per cent efficiency. Similarly, the total of the shaded areas, or 6800 sec.-ft. mo., equivalent to 9,720,000 kw.-hr., is required from storage or auxiliary plant as the case may be.

In this example the flow demand is constant throughout the year; but, in many instances, requirements are such that the demand in certain seasons is greater than in others. This may be caused by fluctuating market requirements, variations in head due to draft for storage, or other circumstances, as will be shown later.

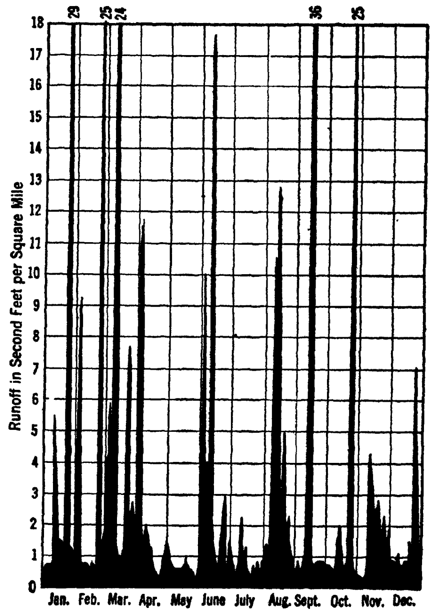

FIG. 47.—Hydrograph of Perkiomen Creek Near Frederick, Pa. 1904.

With a full storage reservoir at the plant on May 15, the deficient flow from that date to Aug. 22 would require a draft on storage of 5420 sec.-ft. mo. to

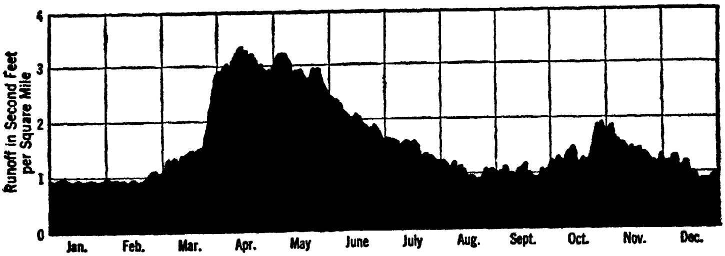

FIG. 48.—Hydrograph of the Richlieu River at Fort Montgomery, N. Y. 1904.

provide the flow demand of 3000 sec.-ft. Between Aug. 22 and Sept. 15, the excess of flow of 1400 sec.-ft. mo. would reduce the reservoir depletion from 5420 to 4020 sec.-ft. mo. On Oct. 1 the depletion would be 4020 + 150 = 4170

sec.-ft. mo.; on Oct. 15, 4170 − 860 = 3310 sec.-ft. mo.; and on Dec. 1, 3310 + 1240 = 4550 sec.-ft. mo.

It is therefore seen that, on Aug. 22, the reservoir would have been drawn to its fullest extent, or 5420 sec.-ft. mo., and this represents the capacity

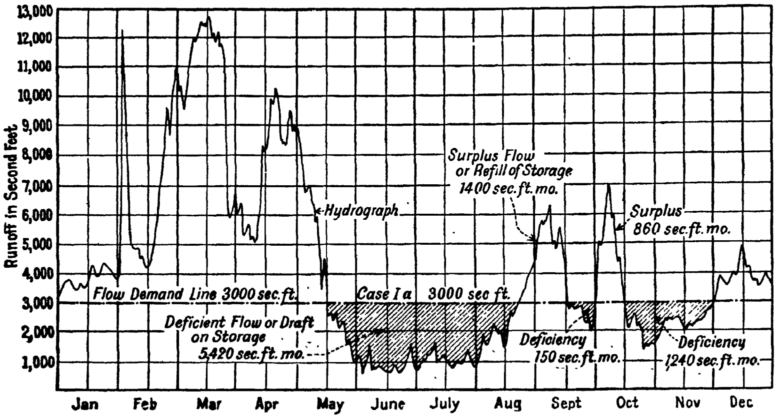

FIG. 49.—Daily Hydrograph of Typical River Showing Storage Requirements.

required to regulate the flow to 3000 sec.-ft. for the year 1918. If, between Dec. 1 and the following period of deficient flow, there are 4550 sec.-ft. mo. of excess flow above the 3000 sec.-ft. demand line, the reservoir will again become full in preparation for service during the next dry season.

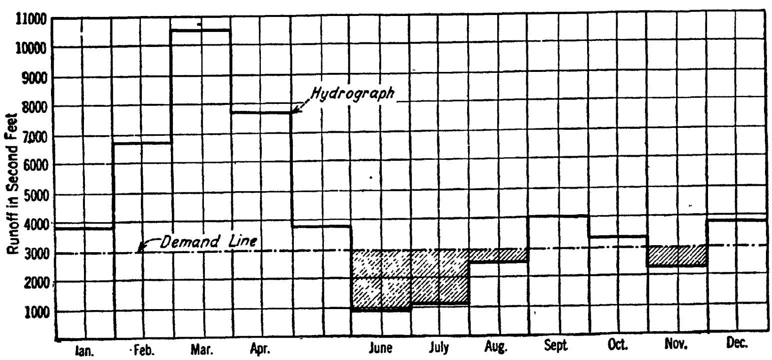

FIG. 50.—Monthly Hydrograph of Typical River for the Year 1918.

Figure 50 is a monthly hydrograph of Typical River for the same period covered by Fig. 49. The use of monthly hydrographs to determine storage requirements over short periods is likely to lead to considerable error, as will be noted by a comparison of the shaded areas of Figs. 49 and 50 indicating required storage capacities of 5420 and 4600 sec.-ft. mo. respectively. Fig. 49

fails to show the deficient flows of May, September, and October, and the deficiency in August is also incorrectly indicated. It will be observed that, when the monthly flow is entirely below or above the demand line, the average monthly flow can be used without error.

For Case *Ib*, with neither pondage nor storage, the stream flow is reduced to usable flow from Curve *AFIGC* of Fig. 46, and plotted as shown by the dotted line in Fig. 51. The shaded area then represents the reduction in output due to lack of pondage. To provide full output at all times, storage must be provided to regulate the flow to 4620 sec.-ft. as that corresponds to the peak demand. Storage requirements are then computed to the 4620 sec.-ft. demand line instead or to the 300 sec.-ft. line of the previous example. Thus it is seen, that, in this case, the lack of a comparatively small pond at

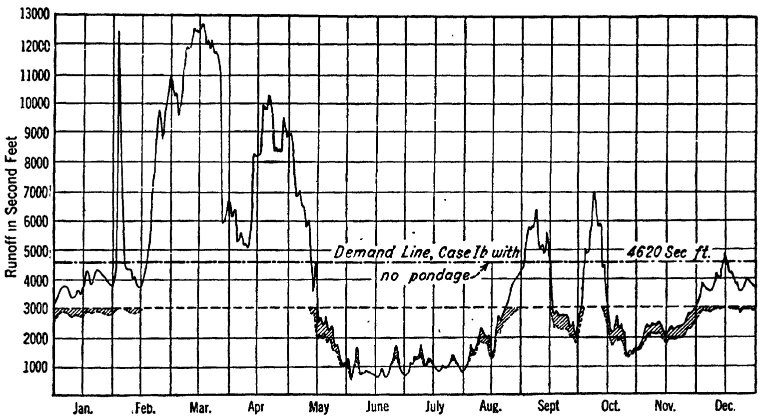

Fig. 51.

the power house makes it necessary to provide about double the storage capacity for continuous output.

As a representative example of extreme non-uniform flow demand, let Fig. 49 represent a hydrograph of the flow from the area intermediate between the development and a distant storage reservoir. Let Fig. 52 be the hydrograph of the runoff from the area above the reservoir. The runoff from the intermediate area is sufficient for all requirements except during the periods indicated by the shaded area in Fig. 49, and this deficiency of flow is laid off as the variable flow demand in Fig. 52. The total hatched area of Fig. 52, therefore, indicates the flow required from the area above the reservoir, and the double-hatched area the deficiency, or storage requirements.

A reservoir can be operated to serve to best advantage only one point on the stream. In the last example it will be noted that a development immediately below the reservoir would be provided with a discharge of an extremely fluctuating nature and entirely unsuited to continuous output. If a reservoir is to serve several developments on the same stream, a compromise operation

must be adopted to the end that the greatest benefit will accrue to the system as a whole.

High-head plants on small drainage areas are sometimes admirably suited to supplement low-head plants on large streams having no storage facilities.

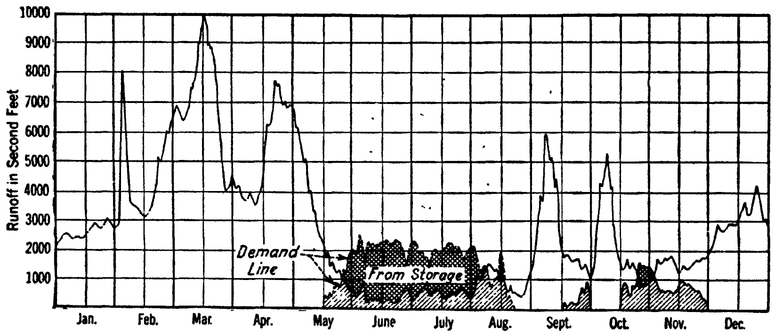

FIG. 52.

The hydrograph indicated in Fig. 49 is for a 33-ft.-head development at a site having a watershed of 2000 square miles.

Figure 53 shows a hydrograph of an adjoining stream draining only 40 square miles but having an available head of 660 ft. One second-foot under

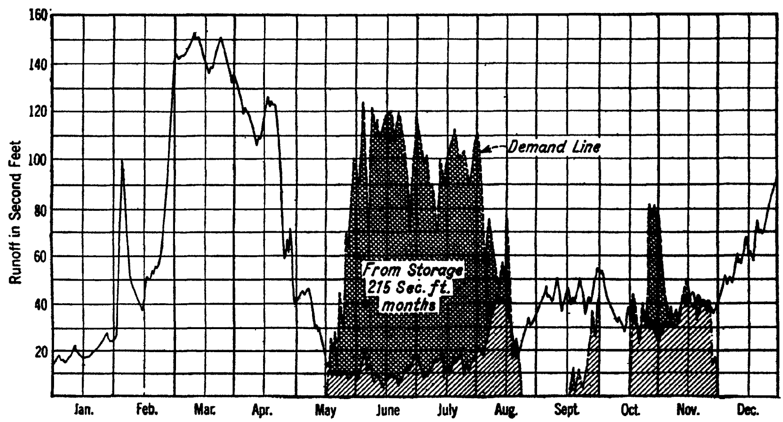

FIG. 53.

the larger head will produce as much power as $\frac{660}{33} = 20$ sec.-ft. on the smaller head. Therefore, at the high-head plant, one-twentieth of a second-foot must be utilized for each second-foot of deficient flow at the low-head plant; and, to supplement the total deficient flow of 6810 sec.-ft. mo. at the low-head plant, only one-twentieth of this amount, or 340 sec.-ft. mo. must be converted

into power under the higher head, properly distributed as indicated in Fig. 53. The required storage is 215 sec.-ft. mo.

The feasibility of this arrangement depends upon the cost of installing and operating a high-head plant of nearly the capacity of the low-head plant, with a 215 sec.-ft. mo. reservoir, as compared with the installation and operation of a 5420 sec.-ft. mo. reservoir to regulate the flow at the low-head plant on the installation and operation of an auxiliary fuel-burning power plant.

If, in the preceding example, the two plants are on the same stream, two revisions must be made in the foregoing calculations. First, the estimated deficiency of flow at the lower plant must be calculated for the area intermediate between the two plants, or 1960 sq. mi. Second, since the water released from the upper area, is usable also at the lower plant, it acts under a total head of 693 ft. Therefore, at the high-head plant, $\frac{33}{693} = \frac{1}{21}$, instead of $\frac{1}{20}$, of the low-head plant deficient flow from 1960 sq. mi. must be released from above the site of the high-head plant and converted into power under the total head of 693 ft.

The hydrograph is perhaps the most easily understood method of indicating diagrammatically the flow of a stream and the regulation of the flow by storage, particularly for those who are not engineers. It is used extensively for explanatory diagrams; but for estimates of power and storage, less work is often involved in other methods.

57. The Mass Curve.—Mass curves are used only to facilitate storage computations. They indicate the total volume of runoff in second-feet months, or other convenient equivalent, during a given period. They are usually made from records of average monthly flows, these records being summed up consecutively and each sum plotted above the corresponding date.

A mass curve of Typical River for the year 1918–1919 is indicated by the heavy line of Fig. 54. The computations for this curve are given in Table XIV. This mass curve indicates that, during the period from Apr. 1, 1918, to Mar. 31, 1919, 52,709 sec.-ft. mo. passed the site.

TABLE XIV

COMPUTATIONS FOR MASS CURVE OF TYPICAL RIVER FOR THE YEAR 1918–1919

Month	Second-feet, Mean Monthly Runoff	Second-feet, Months Accumulative	Month	Second-feet, Mean Monthly Runoff	Second-feet, Months Accumulative
April	7,710	7,710	October	3,300	23,280
May	3,850	11,560	November	2,330	25,610
June	860	12,420	December	3,880	29,490
July	1,040	13,460	January	5,200	34,690
August	2,500	15,960	February	6,000	40,690
September	4,020	19,980	March	12,100	52,790

It is evident that a mass curve plotted from mean monthly flows is correct only at the beginning and end of each month, since the variation in flow during the month is not taken into consideration. For a uniform monthly flow, the

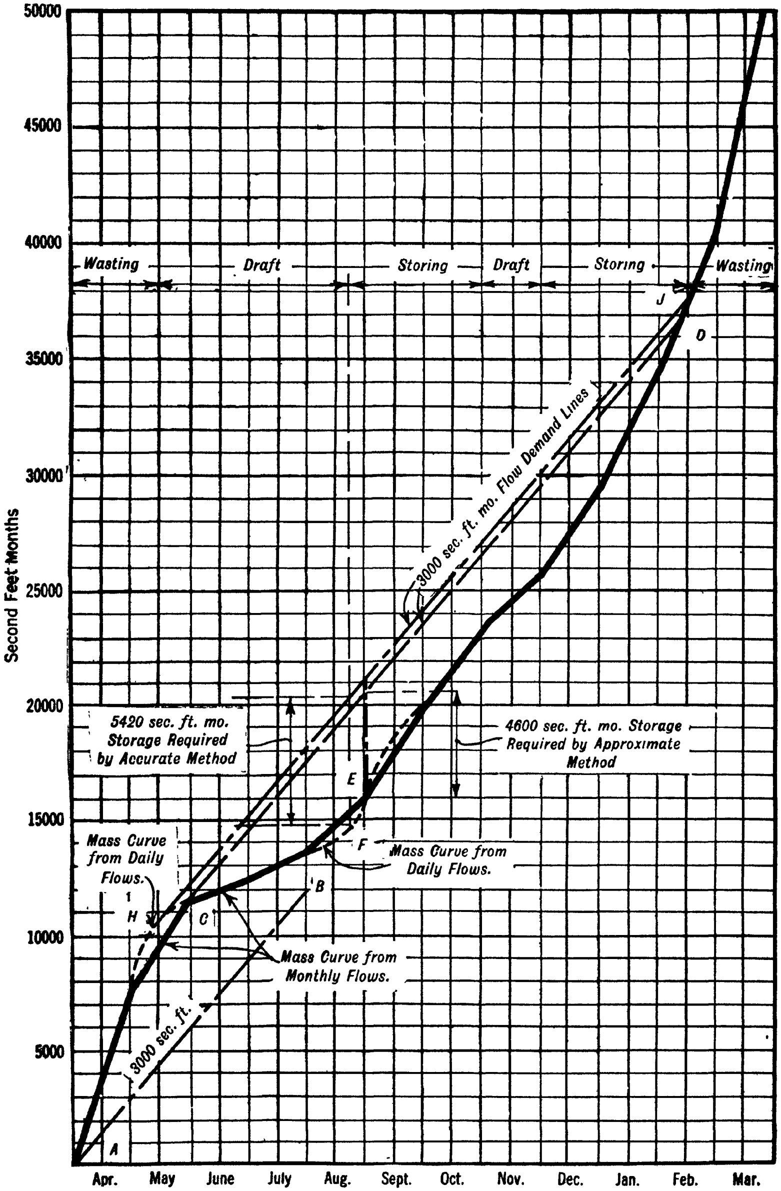

FIG. 54.—Mass Curve of Typical River for the Year 1918–1919, the Lowest of Record Plotted from Table XIV.

mass curve for the month would be a straight line. A true curve for any month would be convex upwards if the flow were greater during the first part of the month. A summation of daily flows, instead of monthly flows, results in a more accurate mass curve; but this involves an excessive amount of work, and daily flows are never used except for critical periods as explained later.

The slope of the curve at any point indicates the rate of flow in second-feet. If the curve is horizontal, the flow is zero. The slope of a line connecting any two points, CD, on the curve is a measure of the average flow during that period, the actual flow being less during the first and greater during the last part of the period. At point C the accumulative flow is 11,560, and at D 36,700 sec.-ft. mo. The difference, or 25,140, is the total flow for the period from C to D, or for 8.38 months. The average flow is therefore $\frac{25{,}140}{8.38} = 3000$ sec.-ft. The average flow can be obtained more easily by drawing a line, AD, through the origin of the curve, parallel to CD. At the end of the first month the ordinate to the line AB is 3000 sec.-ft.

The 3000 sec.-ft. flow demand of Case I*a* of Fig. 46 will be used to explain the determination of storage requirements from the mass curve. As the mass curves for April and May are both steeper than the 3000 sec.-ft. line, AB, the flow during those months is indicated to be in excess of the demand, and no storage is required. The curve for June, however, has a flatter slope. From the point C, draw the line CD parallel to AB representing the demanded flow of 3000 sec.-ft. With a full reservoir at C, draft will occur at all times when the slope of the mass curve is less than the line CD, and the reservoir will be filling when the slope is greater. The depletion of storage at any time is measured by the ordinates between the curve and the line, the reservoir being full at both points C and D. The greatest draft from storage, as measured by the length of the maximum intercepted ordinate at E, is 4600 sec.-ft. mo.

Monthly mass curves, like monthly hydrographs, are subject to errors because of their representing average rather than actual monthly flows. The errors occur at the beginning and at the end of reservoir drawdown. The dotted lines in Fig. 54 indicate the plotting of daily flows for the months of May, August, and September. It will be noticed that the slope of the correct mass curve during the latter part of May is less than the demand line, beginning with the 15th of the month, or at H, where a new demand line would be tangent to the corrected curve. The maximum ordinate is now at F and shows the correct greatest draft to be 5420 sec.-ft. mo.

If the flow-demand line does not intersect the mass curve, as at J, the reservoir will not be filled again. If the reservoir is very large, the time interval between the points H and J may be several years.

To determine if the reservoir would be full at H, a mass curve must be drawn for the preceding year and investigated in the same manner.

The flow-demand line may be curved if the seasonal power demand is not constant. In the preceding example, the storage is at the plant. For remote storage the use of the mass curve is not as adaptable as other methods.

58. The Duration Curve.—Figs. 55 and 56 show duration curves for Typical River of the twenty years of record and for the minimum year of record, respectively. Each point on a duration curve indicates the percentage of time, during the period under consideration, that the flow was equal to a greater than the given discharge. Duration curves may be computed from mean monthly flows or daily flows. Those computed from mean monthly flows are only approximate, because in each month there is a considerable variation in flow not indicated in the duration curve. Variations in flow during twenty-four hours are usually negligible. Fig. 55 shows the error involved in the use of mean monthly flows for the duration curve of Typical River.

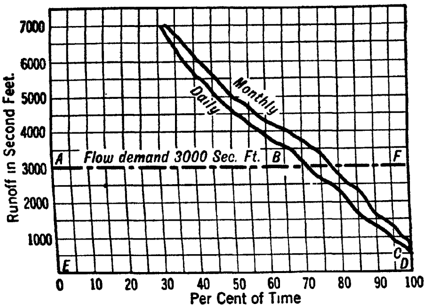

FIG. 55.—Monthly and Daily Duration Curves of Typical River for the 20 Years of Record.

Computations involving the use of daily flows require the consideration of thirty times as many items as those made for mean monthly flows. The time consumed in the use of daily flows has led many engineers to use monthly duration curves exclusively, probably without a realization of the errors involved. Such errors will usually range from 5 to 15 per cent, depending upon the characteristics of the stream and the extent of utilization of the flow. Typical River is an average stream. The difference between the daily and monthly curves would probably be negligible for steady streams similar to the Richelieu River indicated in Fig. 48; but greater than that shown in Fig. 55 for very flashy streams.[1]

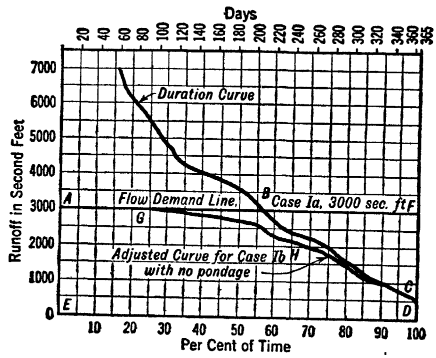

FIG. 56.—Daily Duration Curve of Typical River for Year 1918, the Lowest of Record from Table XV.

Computations involved in the construction of the duration curve of Fig. 56 are given in Table XV. Rates of flow, differing by convenient amounts, are set up in Col. 1. In Col. 2 is indicated the number of times the daily flow equaled a value between the corresponding flow in Col. 1 and the flow next below. During the year, a daily flow between 1000 and 1100 sec.-ft. occurred 10 times.

[1] For other comparisons of daily and monthly duration curves, see News-Record, Vol. 87, p. 250, and Ninth Annual Report of New York State Conservation Commission, 1919.

TABLE XV

COMPUTATIONS FOR DAILY DURATION CURVE OF TYPICAL RIVER FOR THE YEAR 1918, THE MINIMUM OF RECORD

Plotted in Fig. 56

1 Second-feet Runoff	2 Number of Days	3 Days Equaled or Exceeded	4 Per Cent of Time
500	1	365	100
600	12	364	99.7
700	8	352	96.4
800	6	344	94.2
900	9	338	92.6
1,000	10	329	90.1
1,100	6	319	87.4
1,200	7	313	85.7
1,300	3	306	83.8
1,400	9	303	83.0
1,600	13	294	80.5
1,800	6	281	77.0
2,000	14	275	75.3
2,200	21	261	71.5
2,400	14	240	63.7
2,600	10	226	61.9
2,800	10	216	59.2
3,000	5	206	56.4
3,200	5	201	55.0
3,400	18	196	53.7
3,600	11	178	48.7
3,800	17	167	45.7
4,000	13	150	41.1
4,200	13	137	37.5
4,400	10	124	34.0
4,600	2	114	31.2
4,800	8	112	30.7
5,000	13	104	28.5
5,500	13	91	25.0
6,000	14	78	21.4
6,500	4	64	17.5
7,000	5	60	16.5
7,500	0	55	15.1
8,000	6	55	15.1
8,500	10	49	13.4
9,000	3	39	10.7
9,500	5	36	9.9
10,000	8	31	8.5
10,500	4	23	6.3
11,000	1	19	5.2
11,500	5	18	4.9
12,000	9	13	3.6
12,500	4	4	1.1
	365		

A summation of occurrences is tabulated in Col. 3. It shows that a flow of 1000 sec.-ft. was equaled or exceeded on 329 days during the year. Col. 3 is then reduced to per cent of time and recorded in Col. 4. Cols. 1 and 4 are used to plot the duration curve. For a duration curve of a one-year period,

Col. 2 will total 365 days and Cols. 1 and 3 may be used, if desired, to indicate the number of days during the year a given flow was exceeded, as indicated in Fig. 56. For a longer period than a year, Col. 2 will total more than 365 days; but a duration curve for the total years of record may be used to represent an average year and, if days are desired at the bottom of the diagram, percentages are simply multiplied by 365.

Each square in Figs. 55 and 56 is $\frac{500 \times 5 \times 365}{100} = 9130$ sec.-ft. days or 300 sec.-ft. mo.

A flow demand of 3000 sec.-ft. or 36,000 sec.-ft. mo. per year has been indicated in Figs. 55 and 56. This demand corresponds to Case I*a* of Sec. 52. The total area *ABCDE*, below the demand line and the duration curve, or 29,200 sec.-ft. mo. for the minimum year and 31,600 sec.-ft. mo. for the average year, represents the total natural flows available for power during the year. From Eq. (16) of Sec. 50, these correspond to 41,700,000 kw.-hr. and 45,200,000 kw.-hr., respectively, under a productive head of 33 ft. and efficiency of 70 per cent.

The areas *BFC*, or 6800 sec.-ft. mo. in the minimum year, and 4400 sec.-ft. mo. in the average year, equivalent to 9,720,000 kw.-hr. and 6,280,000 kw.-hr. respectively, are a measure of the power required from storage or auxiliary plants as the case may be. The curves also show that natural flow sufficient for full output is available 56.5 per cent of the time, or 206 days, in the minimum year, and 71 per cent or 259 days, in the average year.

The area *BFC* is the total discharge from storage required to supplement the natural flows; but it is no indication of the required capacity of the reservoir, as the reservoir may be filled and emptied more than once during the year. This feature is more clearly indicated in Sec. 56 which shows that for 1918, the minimum year, a capacity of 5420 sec.-ft. mo. was required to supply the total demand of 6800 sec.-ft. mo.

For Case I*b*, with neither pondage nor storage, the steam flow, as indicated by the duration curve, is reduced to usable flow from Curve *AFIGC* of Fig. 46, and plotted as shown by the line *AGHC* of Fig. 56. The area *GHCB* then represents the reduction in output due to lack of pondage, and the area *FGBHC* is the output required from storage or auxiliaries.

For the case of remote storage but no pondage, the regulated flow at the plant may be computed by analytical methods. If the reservoir is not regulated for the plant in question, or if it is of capacity sufficient only for partial regulation, the flow at the plant can be plotted as a duration curve, then adjusted for lack of pondage as previously described, and the output computed.

59. Analytical Methods.—Analytical methods are adaptable only to cases where adequate pondage is available. Average monthly flows must be used exclusively because a consideration of daily flows would result in an endless amount of labor. An example of calculation by analytical methods will be given for a plant having no storage, and one for a plant with storage. A constant flow demand of 3000 sec.-ft., corresponding to Case I*a* of Sec. 52, will be used in both examples.

No Storage.—If, in any month, the average flow is equal to or greater than the flow demand of 3000 sec.-ft., it does not follow that without storage, all power demands will be met. In Table XII, the average monthly flow of Typical River during May, 1918, is given as 3850 sec.-ft. By reference to Table XI, however, it is seen that the flow during part of that month was considerably less than the flow demand of 3000 sec.-ft., resulting in a deficiency in output. Moreover, if the average flow is less than the flow demand, it is not necessarily true that, without storage, all of the flow can be converted into power, because it is quite possible that, during some part of the month, the flow was in excess of the flow demand and some waste necessary. This

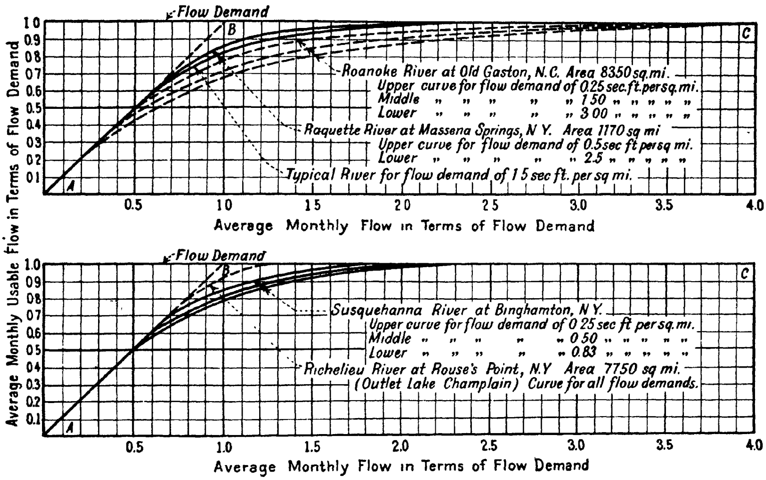

FIG. 57.—Typical Use Curves.

feature has led to the adoption of *use curves*, samples of which are shown in Fig. 57.

Use curves show the relation between average monthly flow and probable usable flow, both expressed either in terms of the flow demand or of second-feet flow. The use curve of Typical River is shown in both ways, the latter indicated in Fig. 58. If, for instance, the average flow of Typical River for a given month was 1.2 times the flow demand, or 3600 sec.-ft., the probable average usable flow during that month is indicated in Figs. 57 and 58 to be 0.88 times the flow demand or 2640 sec.-ft.

The use curves of Fig. 57 were plotted from an analysis of the daily records of a number of years of each stream. It will be noticed that, for very steady streams such as the Richelieu (see Fig. 48), the use curve approaches the line *ABC* which corresponds to the case where the flow is absolutely constant during each month. The construction of a use curve involves considerable labor and, for approximate computations, an existing curve of a similar stream may be used without great error.

The average monthly flows of Table XIII have been reduced to approximate average monthly usable flows in Table XVI by means of the use curve of Fig. 58. The average uasble flow for the twenty-year period is seen to be 2630 sec.-ft. or 31,560 sec.-ft. mo. per annum. This corresponds to the 31,600 sec.-ft. mo. in the average year found by the duration curve method of Sec. 58.

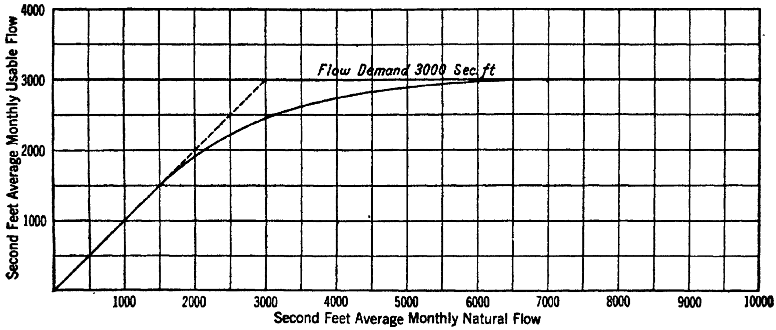

FIG. 58.—Use Curve of Typical River.

TABLE XVI

COMPUTATIONS FOR AVERAGE USABLE FLOW, TYPICAL RIVER

Flow Demand, 3000 Sec.-ft. No Storage

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Average
1900	2740	3000	3000	3000	3000	1210	910	2070	2670	2740	2210	2740	2430
1901	3000	3000	3000	3000	2900	1380	1360	2160	2730	2850	2420	2800	2550
1902	2800	3000	3000	3000	2780	2470	1420	2520	2840	3000	2960	3000	2730
1903	3000	2720	3000	3000	3000	2800	2630	2590	3000	3000	3000	3000	2890
1904	3000	2820	3000	3000	2500	1190	2570	2900	2850	2900	3000	3000	2720
1905	3000	3000	3000	3000	3000	2470	1020	2200	2760	2640	2100	2740	2580
1906	3000	3000	3000	3000	3000	2220	1580	2200	2740	3000	2560	3000	2690
1907	3000	3000	3000	3000	3000	970	2340	2450	2830	3000	2930	2880	2700
1908	3000	3000	3000	3000	2320	1070	2160	2610	2870	2930	3000	2850	2650
1909	3000	3000	3000	3000	2940	1430	990	1950	2900	2620	2160	2900	2490
1910	2840	3000	3000	3000	3000	1510	1600	1970	2970	2700	3000	3000	2630
1911	3000	3000	3000	3000	2700	2000	1700	2820	2870	3000	2670	3000	2730
1912	2980	2800	3000	3000	2800	1040	1460	2670	2820	2940	3000	2620	2590
1913	3000	3000	3000	3000	3000	920	1700	2420	2760	2760	2920	2950	2620
1914	2970	2670	3000	3000	2930	2620	1370	2320	2880	3000	2760	2940	2700
1915	2880	3000	3000	3000	3000	850	1240	1990	2650	2900	2220	2820	2460
1916	3000	3000	3000	3000	3000	1630	1480	2720	2880	2970	3000	2450	2680
1917	2940	3000	3000	3000	3000	2350	1630	2760	2780	2810	2830	2940	2750
1918	2700	3000	3000	3000	2700	860	1040	2200	2750	2550	2120	2720	2390
1919	2930	2970	3000	3000	3000	1800	1540	2380	2800	3000	2880	2970	2690
										General average,			2630

From Table XVI, the average usable flow for the minimum year, 1918, is 2390 sec.-ft. or 28,680 sec.-ft. mo., corresponding to the 29,200 sec.-ft. mo. found by the hydrograph and duration curve methods for that year.

The deficiency, to be made up by auxiliaries, is the difference between the second-feet months of usable flow and the flow demand of 36,000 sec.-ft. mo. per annum, or 4440 sec.-ft. mo. in the average year, and 7320 sec.-ft. mo. in the minimum year. The usable flows and deficiencies may be converted into units of energy by Eq. (16) of Sec. 50, as previously explained.

With Storage at the Plant.—With storage at the plant, the problem is complicated by the fact that the flow demand is not constant, because of the necessary reduction in head during draft. For simplicity, however, a constant flow demand of 3000 sec.-ft. will be used in the following example as in the preceding one.

Table XVII indicates the usual approximate analytical method for determining storage requirements when the reservoir is at the plant. The method is obvious. It shows the greatest draft on storage for Typical River in 1918 to be 4600 sec.-ft. mo.

This corresponds to the use of the monthly hydrograph of Fig. 50 for computing storage requirements and involves the error, as explained in Sec. 56, that the latter part of May and the first part of September are deficient in flow, as indicated in Fig. 49, notwithstanding the fact that the average monthly flows of these months are considerably above the flow demand of 3000 sec.-ft. Such errors become smaller as the period of deficient flow becomes longer, and for relatively large reservoirs, depleted for long periods, the error will be negligible. For short periods of depletion, those months in which the flow is partly below and partly above the flow demand must be examined in detail.

TABLE XVII

TYPICAL RIVER, YEARS 1918 AND 1919, APPROXIMATE COMPUTATIONS FOR STORAGE REQUIREMENTS, RESERVOIR AT PLANT

Second-feet Months

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1918												
Natural stream flow..	....	....	.....	.....	3850	860	1040	2500	4020	3300	2330	3880
Draft from storage...	....	....	.....	.....	0	2140	1960	500	.....	....	670	
Storage refill.........	....	....	.....	.....	....	.....	.....	....	1020	300	.....	880
Regulated stream flow.	....	....	.....	.....	3850	3000	3000	3000	3000	3000	3000	3000
Reservoir depletion...	....	....	.....	.....	0	2140	4100	4600	3580	3280	3950	3070
1919												
Natural stream flow..	5200	6000	12,100	10600	9600	1860	1550	2800	4300	6200	4900	
Draft from storage....	....	....	0	0	0	1140	1450	200	.....	....	0	
Storage refill.........	2200	870	.....	.....	....	.....	.....	....	1300	1490	.....	
Regulated stream flow.	3000	5130	12,100	10600	9600	3000	3000	3000	3000	4710	4900	
Reservoir depletion...	870	0	0	0	0	1140	2590	2790	1490	0	0	

With Storage Remote from Plant.—If the storage is remote from the plant, Table XVIII indicates the usual method of calculating storage requirements. In this example it has been assumed that one-tenth of the flow of the previous example is available from the area below the reservoir, and nine-tenths from the area above. These are indicated in Lines 1 and 4. The flow past

the reservoir, when the reservoir is partly depleted, must be such that the total flow at the plant equals the flow demand, and the draft on storage or refill is the algebraic difference between Lines 4 and 2.

TABLE XVIII

TYPICAL RIVER, YEARS 1918 AND 1919, APPROXIMATE COMPUTATIONS FOR STORAGE REQUIREMENTS, RESERVOIR REMOTE FROM PLANT

Second-feet Months

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1918												
Natural flow below reservoir	....	....	......	....	3465	774	936	2250	3618	2970	2097	3492
Flow past reservoir	....	....	......	....	0	2226	2064	750	0	30	903	0
Total flow at plant	....	....	......	....	3465	3000	3000	3000	3618	3000	3000	3492
Natural flow above reservoir	....	....	......	....	385	86	104	250	402	330	233	388
Draft from storage	....	....	......	....	0	2140	1960	500	.....	....	670	....
Storage refill	....	....	......	....	0	....	....	....	402	300	.....	388
Reservoir depletion	....	....	......	....	0	2140	4100	4600	4198	3898	4568	4180
1919												
Natural flow below reservoir	4680	5400	10,890	9540	8640	1674	1395	2520	3870	5580	4410	5310
Flow past reservoir	0	0	0	0	170	1326	1605	480	0	0	0	0
Total flow at plant	4680	5400	10,890	9540	8810	3000	3000	3000	3870	5580	4410	5310
Natural flow above reservoir	520	600	1,210	1060	960	186	155	280	430	620	490	590
Draft from storage	....	....	......	....	....	1140	1450	200	.....	....	.....	....
Storage refill	520	600	1,210	1060	790	....	....	....	430	620	490	590
Reservoir depletion	3660	3060	1,850	790	0	1140	2590	2790	2360	1740	1250	660

It will be noticed that, for this case, the 1918 draft on storage is greater than for the previous example, because, in this case, an excess flow of 618 sec.-ft. mo. from the lower area was wasted past the site in September when the reservoir was in need of replenishment.

60. Incomplete Storage.—If the available storage is insufficient to regulate the stream so as to provide the flow demand continuously during years of low runoff, the reservoir may be operated in either of two ways, depending upon the results to be accomplished.

Case 1.—Regulation to provide maximum possible total energy output.

Case 2.—Regulation to provide maximum possible primary energy output.

Case 1.—Fig. 59 is a hydrograph of the low-season flow of Typical River in 1918, the minimum year of record. The flow demand is 3000 sec.-ft. Regulation of the flow during this period with complete storage is indicated in Fig. 49, and Sec. 56 showed that 5420 sec.-ft. mo. of storage would be required. If, however, only 3500 sec.-ft. mo. of storage is available, and maximum possible energy output is desired, irrespective of its distribution, the plant must be operated to its demand of 3000 sec.-ft. during all periods in which flow from storage is available. In other words, the reservoir must be drawn down as quickly as possible in order to be in a condition to receive and store, in so far as possible, all flows in excess of the flow demand. Fig. 59 shows by the full

lines *ABC*, that operation under these conditions would have depleted the reservoir in 1918 by July 15. Operation at less than full demand, when the reservoir contains water, creates the risk that, if the season develops into one of relatively good flow, the reservoir may not be emptied and incomplete use of storage may result. In dry years, this scheme of operation provides very low output during the latter part of the period of low flow, and hence the output is suitable only for secondary power.

Case 2.—If the maximum possible primary output is desired, all years of record must be studied and an estimated hydrograph constructed to indicate the minimum probable low-water period. For this period the maximum possible continuous or primary output with the available storage must be deter-

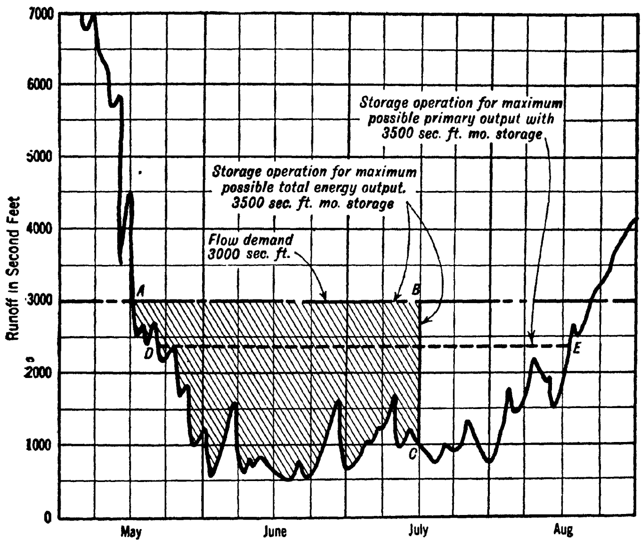

FIG. 59.—Daily Hydrograph of Typical River for the Minimum Year of 1918 Showing Incomplete Storage.

mined, as indicated by the dotted line *DE* of Fig. 59, which shows that, with 3500 sec.-ft. mo. of storage, the flow can be regulated to a maximum continuous output of 2400 sec.-ft. in the driest year. This represents the limit of primary output, and the reservoir must be operated during the low season of all years to provide this amount only.

In years during which the low-season flow is relatively good, it would be possible to deplete the 3500 sec.-ft. mo. of storage to obtain a continuous utilization in excess of 2400 sec.-ft., and in many years to obtain the full demanded flow of 3000 sec.-ft.; but, as advance information on future flows is never available, the river cannot be regulated to a greater flow than the 2400 sec.-ft. maximum possible dryest-season continuous flow, for fear that a deficiency approaching the minimum may occur at any time, and the latter

part of the dry period find the reservoir empty and primary power output interrupted for lack of water, as indicated by the full lines, *ABC*. The proper operation of the reservoir for this case requires incomplete utilization of storage in wet years, resulting in a lower total energy output.

In other than dry seasons, however, the reservoir could be operated to provide greater continuous output than 2400 sec.-ft., provided that the reservoir at such times contained sufficient water to meet the deficiency which a study of all existing records would indicate to be a maximum for that season.

61. Reservoir-depletion Problems.—In many cases it is required to determine the reservoir capacity which will insure reasonably uninterrupted demanded flow. Failure to fulfill primary power contracts is objectionable from a business standpoint, even though penalties and damage suits do not result. Interruptions due to inadequate storage provisions occurring once in ten or twenty years, may, in some cases, be preferable to the expenditures necessary for larger storage. On the other hand, for more important markets it may be desirable to provide storage of such capacity that depletion will not be possible except on an average of once in 50 or 100 years.

The application of the theory of probabilities to this problem was first made by Allen Hazen.[2] The general theory of probability curves is given in Sec. 86. In Table XIX the reservoir-depletion frequency of Typical River, for a flow demand of 3000 sec.-ft., is computed. The maximum storage

TABLE XIX

COMPUTATIONS FOR RESERVOIR-DEPLETION FREQUENCY, TYPICAL RIVER

	Second-feet Month Storage Required					Second-feet Month Storage Required			
1	2	3	4	5	1	2	3	4	5
Year	Actual	In Order of Magnitude	n	p	Year	Actual	In Order of Magnitude	n	p
1900	4930	780	20	97.5	1910	2830	3170	10	47.5
1901	3960	1830	19	92.5	1911	2590	3210	9	42.5
1902	2590	2390	18	87.5	1912	4030	3310	8	37.5
1903	780	2590	17	82.5	1913	3690	3690	7	32.5
1904	2760	2590	16	77.5	1914	2390	3960	6	27.5
1905	3310	2670	15	72.5	1915	5160	4030	5	22.5
1906	2670	2760	14	67.5	1916	3170	4700	4	17.5
1907	2890	2830	13	62.5	1917	1830	4930	3	12.5
1908	3210	2890	12	57.5	1918	5420	5160	2	7.5
1909	4700	2990	11	52.5	1919	2990	5420	1	2.5

requirements for each year of record are computed and recorded in Col. 2. These are then arranged in their order of magnitude in Col. 3, and the probable frequency percentage calculated from the following equation:

$$p = \frac{100(n - 0.5)}{m};$$

[2] See Reference 1, Sec. 63.

where p = the percentage of all future years in which the required storage capacity will equal or exceed the amount recorded in Col. 3, expressed as a whole number;
n = the serial number of the order of magnitude as indicated in Col. 4;
m = the number of years of record.

Columns 5 and 3 are used to plot the frequency points of Fig. 60, which

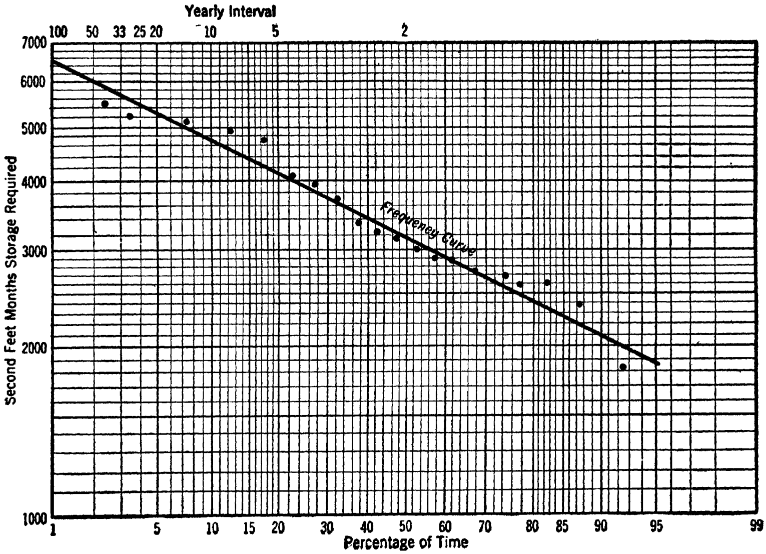

FIG. 60.—Reservoir Depletion Frequency Curve, Typical River.

serve to indicate the general direction of the frequency curve. The yearly interval scale of Fig. 60 is computed from the following equation:

$$I = \frac{100}{p},$$

where I is the yearly frequency interval or the number of years during which a given capacity of reservoir would be required once.

From the frequency curve it is seen that, for Typical River and a flow demand of 3000 sec.-ft., a 5300 sec.-ft. mo. capacity of reservoir would probably be depleted, on an average, once in about twenty years, and a 6000 sec.-ft. mo. reservoir once in about fifty years.

62. Effect of Other Plants on Output.—If two adjacent plants on the same stream utilize equal flow and operate on the same market, the lower plant requires no pondage, as its varying demand coincides with the discharge from the upper plant. If the plants are some distance apart, so that it requires several hours for the discharge from the upper plant to reach the lower, the demand at the lower plant may be at extreme variance with the flow, as in the

simple case of two plants operating solely on a uniform day load and at a distance apart such that the flow from the upper plant requires twelve hours to reach the lower. If the lower plant has no pondage, it can develop no power during the low-water season as, at such times, the total flow of the stream would pass the plant during the night when no power is demanded.

It is obvious that, for a plant having inadequate pondage, the effect of the influence of upper plants on the flow is very difficult to analyze, as it involves a consideration of relative capacity and head, a comparison of the load curves, and estimates of the mean velocity of flow of the river at various stages.

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CHAPTER IX

HYDRAULICS

By William P. Creager

64. Flow through Orifices and Short Tubes.—Typical examples of orifices and short tubes are given in Fig. 61. A knowledge of the laws of the flow of water through them is necessary in determining the discharge through sluiceways and the entrances to conduits. If the entrance is not properly shaped, a contraction of the jet occurs as in Sketches *a*, *c* and *h* and the area of the jet is not as great as the area of the orifice or tube. For properly rounded approaches to orifices, as in Sketches *b* and *e*, and in the constant diameter short tubes shown in Sketches *d*, *f* and *g*, the diameter of the jet

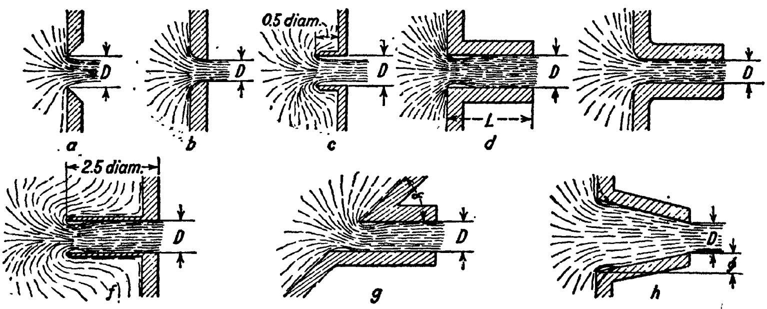


Fig. 61.

is equal to the area of the orifice or tube. In the case of short tubes without rounded entrances, the contraction does occur; but the jet expands again [1] as indicated, a partial vacuum occurring just inside the entrance.

Let H = the head of water in feet on the center line of a freely flowing orifice or tube, or the difference in water level for a submerged orifice or tube;

a = the area in square feet of the orifice or tube;

v = the theoretical velocity in feet per second corresponding to head H;

g = the acceleration of gravity = 32.2 ft. per second;

[1] With certain exceptions as explained later.

Q = the discharge in cubic feet per second;
C' = the coefficient of contraction, or ratio of the area of the jet to the area of the orifice or tube;
C'' = the coefficient of friction;
C = the coefficient of discharge.

The general equation for the velocity of spouting water is:

$$v = \sqrt{2gH}. \qquad (20)$$

Considering friction, the actual velocity due to head H is

$$v = C'\sqrt{2gH}.$$

The discharge is equal to the product of the actual velocity and the area of the jet, or, since the area of the jet is $C''a$,

$$Q = vC''a,$$

or

$$Q = C'C''a\sqrt{2gH}.$$

In experiments conducted to determine the discharge through orifices and tubes, the coefficient of friction, C', and the coefficient of contraction, C'', are combined, and the general equation for discharge is:

$$Q = Ca\sqrt{2gH}. \qquad (21)$$

The value of the coefficient of discharge, C, varies with the shape of the orifice or tube. Average experimental values of the coefficient of discharge, C, for use in Eq. (21), for several types of orifices and tubes, are given in Table XX.[2] These orifices and tubes are circular unless otherwise described. According to Bovey and other authorities on orifices in thin plates (Fig. 61a), the value of C for circular orifices is about 2 per cent less than for square orifices; from 3 to 4 per cent less than for rectangular orifices having a ratio of length to height of 4; and from 5 to 7 per cent less than for rectangular orifices having a ratio of length to height of 10. A similar relation probably exists for tubes having square-cornered entrances.

The coefficient C is *not greatly* affected by submergence.

Coefficients for short tubes apply only to heads less than about 40 ft. For higher heads the expansion heretofore explained does not occur and the coefficients C approach those for orifices of similar type.

The expansion of the jet within the short tubes may not occur if they are not submerged and if sufficient friction is lacking. In other words, the jet, after contracting, may shoot through the tube without touching its sides, if under a high head; and even if the trajectory is such that the jet strikes the bottom of the tube, the expansion will not occur if the friction along the bottom is insufficient.

The shape of the jet from a sharp-cornered round orifice, Fig. 61a, between the orifice and the *vena contracta*, or section at which the area of the jet

[2] Head large in comparison with size of orifice or tube. See later discussion.

TABLE XX

COEFFICIENTS OF DISCHARGE, C, THROUGH ORIFICES AND TUBES FOR EQ. (21). CIRCULAR EXCEPT AS NOTED

Figure	Type	Coefficient C
61*a*.	Orifice in thin plate	0.60
61*b*.	Rounded orifice	0.97
61*c*.	Inwardly projecting orifice	0.50
61*d*.	Short tubes with sharp-cornered entrances,*	
	Values of $\frac{L}{D}$	
	0	0.60
	0.25	0.63
	0.50	0.67
	0.75	0.72
	1.00	0.76
	1.50	0.79
	2.50	0.80
	3.50	0.80
61*e*.	Short tube with rounded entrance	0.97
61*f*.	Inwardly projecting tube with sharp-cornered entrance	0.72 to 0.80
61*g*.	Inclined short tube with sharp-cornered entrance,†	
	Values of α	
	90°	0.82
	80°	0.80
	70°	0.78
	60°	0.76
	50°	0.75
	40°	0.73
	30°	0.72
61*h*.	Convergent short tube,‡ Sharp-cornered entrance	
	Values of ϕ	
	0°	0.82
	5.75°	0.94
	11.25°	0.92
	22.50°	0.85
61*h*.	Rounded cornered entrance	
	Values of ϕ	
	0°	0.97
	5.75°	0.95
	11.25°	0.92
	22.50°	0.88
	45.00°	0.75

* From experiments by Rogers and Smith on submerged tubes, Eng. News, Vol. 76, p. 827, 1916. The coefficient for $\frac{L}{D}$ of 2.5 and greater has been found by other experimenters to be 0.82.

† According to Weisbach.

‡ H. W. King after Unwin.

becomes constant, is indicated in Fig. 62[3] and should be used for rounded entrances as in Fig. 61*e*.

The coefficients in Table XX have been obtained from experiments under ideal conditions. As an indication of the smaller discharge coefficients obtained for practical cases, there are given in Fig. 63 the results of Stewart's

[3] A. H. Gibson, after Weisbach.

experiments [4] on the discharge through 4.0-ft. square, submerged sluices of different types.

The minimum value of C for each set of experiments is shown. The forms of the entrances to the sluices are given below and shown in Fig. 64.

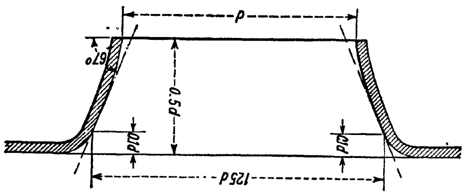


FIG. 62.

Series A, Square entrance;

Series a, Contraction suppressed on bottom;

Series b, Contraction suppressed on bottom and one side;

Series c, Contraction suppressed on bottom and two sides;

Series d, Contraction suppressed on bottom, two sides, and top.

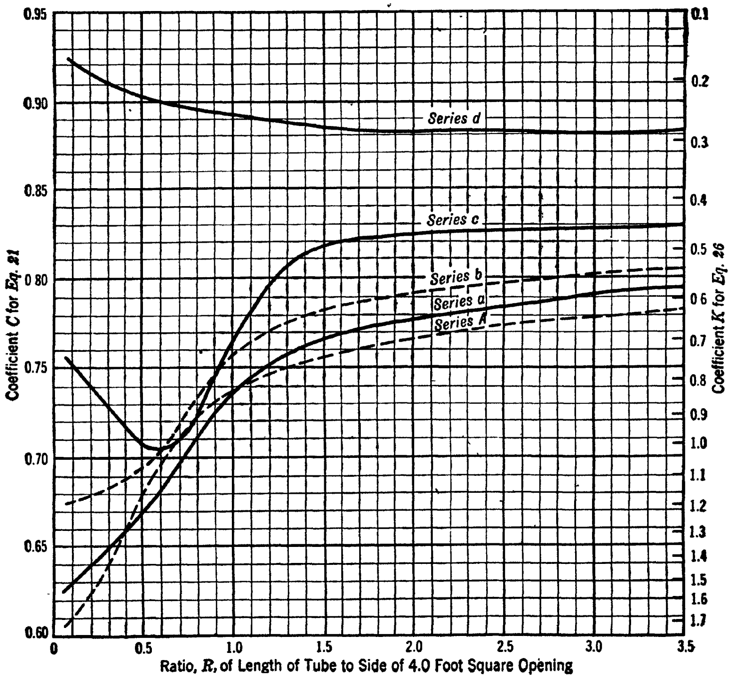


FIG. 63.—Minimum Coefficient of Discharge, C, and Corresponding Coefficients, K, from Stewart's Experiments.

The peculiar shape of the curve for Series c is probably an error in the experiments. The experiments covered heads ranging from 0.05 to 0.30 ft.

[4] "Investigation of Flow through Large Submerged Orifices and Tubes," by C. B. Stewart, University of Wisconsin Bulletin, 216, 1908.

It was found that the coefficient C was least for heads of 0.15, and these losses are the ones indicated in Fig. 63.

In these experiments, Series A corresponds to Item 61d of Table XX, in which the coefficient C varied from 0.60 to 0.80 as compared with Stewart's coefficients of 0.60 to 0.784, which is a reasonably close agreement; but Series d corresponds to Item 61e of Table XX in which C is given as 0.97, as compared with Stewart's variation of 0.925 to 0.882. In the latter case, however, the considerably lower values of Stewart's coefficient were probably caused in part by an imperfect mouthpiece.

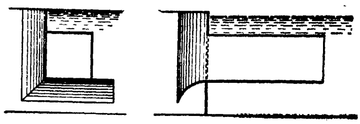


FIG. 64.—Stewart's Sluice, Series b (showing contraction suppressed on bottom and one side).

Velocity of Approach.—If the velocity in the channel of approach were uniform, the actual head on the orifice or tube would in reality be the measured head plus the head corresponding to the velocity of approach. The velocity of approach, however, is not uniform through any section of the channel, being less than the average at the sides and bottom. The velocity directly opposite the orifice is therefore greater than the average, and the effective head on the orifice is the measured head plus a head somewhat greater than that corresponding to the velocity of approach.

Equation (21), corrected for velocity of approach, may be written:

$$Q = Ca\sqrt{2g\left(H + \beta\frac{v^2}{2g}\right)} = Ca\sqrt{2g(H + \beta h_v)}, \quad . \quad . \quad . \quad (22)$$

where v is the average velocity of approach, h_v the head corresponding to this velocity, and β a coefficient which must be determined experimentally. Unfortunately, β is not well known for many types of orifices and may vary between 1.0 and 2.0, depending upon the location and relative size of the orifice.

Rectangular Orifices and Tubes under Low Head.—If the area of the freely flowing orifice or tube is large in comparison with the head, the equation for discharge should be written as follows:

$$Q = \frac{2LC}{3}\sqrt{2g}(H_1^{3/2} - H_2^{3/2}), \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

where H_1 and H_2 are the heads on the bottom and top of the orifice, respectively.

If the orifice is completely submerged, the head is the difference in level between the upper and lower water surfaces, and Eq. (21) applies.

If, in Eq. (23) the top of the orifice is at the water surface, $H_2 = 0$ and the equation reduces to:

$$Q = \frac{2LC}{3}\sqrt{2g}H^{3/2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (24)$$

This is the basic theoretical equation for discharge over weirs.

A. H. Gibson says that Eq. (21) can be used with an error not greater

than 1.0 per cent if the head on the top of the freely flowing orifice is greater than twice the height of the orifice.

65. Discharge through Sluice Gates.—Sluice gates are made in a variety of forms. Many types of sluices, controlled by sluice gates, are in reality short conduits leading through the dam in which skin friction is a large percentage of the total loss. A discussion of the losses through conduits is given hereinafter. If the sluice is short, the discharge may be considered as that through a short tube (Figs. 61*d* to 61*h*, inclusive) or, if the sluice gate is in a thin wall (Figs. 61*a* to 61*c* inclusive), the discharge may be obtained from Eqs. (21) or (22) with the proper coefficient of discharge selected from Table XX, or Fig. 63 according to the details of the sluice-gate opening as explained in Sec. 64.

66. Loss of Heads in Conduits.—The loss of head in conduits may be divided into two general groups:

(*a*) Eddy losses, which are caused by sudden changes in the direction of flow, as at bends, branches, etc., or by sudden changes in velocity due to sudden changes in area as at the entrance, sudden enlargements, valves, etc.

(*b*) Skin friction in straight, uniform conduits.

67. Eddy Losses in Conduits.—It is convenient to measure eddy losses in terms of the velocity head of the flowing water. The velocity head, or head required to produce a given velocity, may be obtained by transposing Eq. (20) of Sec. 64:

$$\text{Velocity head} = h_v = \frac{v^2}{2g} \qquad (25)$$

Then h_f, the head lost at any point in the conduit due to eddies, is:

$$\text{Eddy loss} = h_f = Kh_v = K\frac{v^2}{2g}. \qquad (26)$$

Where K is the coefficient of eddy loss and v is the highest velocity at the point under consideration.

68. Losses at Conduit Entrances.—A discussion of the flow through short tubes and sluices is given in Sec. 64, and the coefficients of discharge C for several types of entrances are indicated. There is a direct relation between the coefficient of discharge for short tubes and the coefficient of eddy loss, K (Eq. (26)), which can be derived as follows:

A drop in pressure or head at the entrance to a conduit is required for two purposes:

(*a*) Velocity head to provide the necessary velocity;

(*b*) Head to overcome friction due to eddies.

Or,

$$H = h_v + h_f = \frac{v^2}{2g} + K\frac{v^2}{2g} = \frac{v^2}{2g}(1 + K).$$

As $Q = av$,

$$H = \frac{Q^2}{2ga^2}(1 + K).$$

Also from Eq. (21),

$$H = \frac{Q^2}{2ga^2} \cdot \frac{1}{C^2}.$$

Combining, we have,

$$K = \frac{1}{C^2} - 1. \qquad (27)$$

The loss of head in short tubes where there is no residual contraction of the jet, as in Figs. 61*d*, 61*e*, 61*f*, and 61*g* and in the sluices used in Stewart's experiments given in Sec. 64, may be assumed to be the same for similar entrances to closed conduits; and values of K derived from the experimental values of C of Sec. 64, are given below.

TABLE XXI

COEFFICIENTS OF EDDY LOSS FOR EQ. (26)

Figure	Type	Coefficient K
61*d*.	Short tube with sharp-cornered entrance	0.56
61*e*.	Short tube with rounded entrance	0.06
61*f*.	Inwardly projecting tube with sharp-cornered entrance*	0.56 to 0.93
61*g*.	Inclined tube with sharp-cornered entrance	
	$\alpha = 90°$	0.49
	$\alpha = 80°$	0.56
	$\alpha = 70°$	0.65
	$\alpha = 60°$	0.73
	$\alpha = 50°$	0.78
	$\alpha = 40°$	0.88
	$\alpha = 30°$	0.93

* Depending upon distance of projection.

Values of K for the sluices used in Stewart's experiments, which are also applicable to losses at entrances of similar type, are given in Fig. 63 and discussed in Sec. 64.

69. Losses at Conduit Intakes.—Losses at the intake racks are usually very small because the velocity must be low to facilitate raking. For clear racks, a safe approximate value of K for use in Eq. (26) may be obtained from the following equation:

$$K = 1.45 - 0.45R - R^2, \qquad (28)$$

where R is the ratio of net to gross area of racks and supports, and v in Eq. (26) is the velocity in the net area of racks and supports. For a usual value of $R = 0.65$ the resulting value of K is 0.74, and for a usual velocity of 2.5 ft. per second in the net area, the loss is 0.072 ft. However, allowance should be made for partial closure of the racks with trash. Twenty-five to 50 per cent of the area of hand-raked racks is frequently obstructed in practical operation where the amount of debris in the water is considerable. This would increase the loss in the above example 0.13 to 0.29 ft.

The head losses in well-designed intakes, with the exception of the loss at the racks and gates, are usually negligible, as changes in area are gradual.

Losses at the head gates correspond to the losses previously given for con-

duit entrances, the worst practical condition being that in which the gate is at the head of an entrance similar to the one indicated in Fig. 61*d*. Usually, however, it is possible to provide economically a flare at the upper end of the intake, so as to produce a gradually increasing velocity between the racks and the gates, with sides and bottom of the intake nearly flush with the gate opening, as in Fig. 237. In such cases there is a decided eddy only at the top of the gate, and the coefficient K is no greater than about 0.5 in Eq. (26) where v is the velocity through the gate.

The gradual increase in velocity between the gate and the normal section of the conduit is made without appreciable loss.

Intakes to open flumes vary greatly in type according to the conditions to be met and the ideas of the designer. Losses at the gates must be estimated by comparison with the nearest type of test model indicated in Fig. 63.

70. Losses at Conduit Bends.—In the present discussion, values of K for use in Eq. (26), in deriving the losses in bends of conduits, are such as to give the loss in excess of that which would occur in a straight pipe of equal length. In other words, normal skin friction is not included.

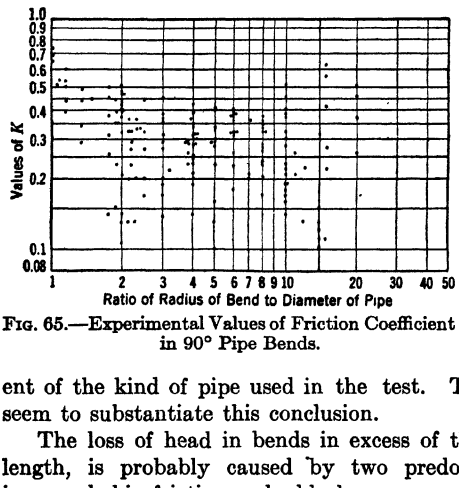


FIG. 65.—Experimental Values of Friction Coefficient in 90° Pipe Bends.

An effort has been made in Fig. 65 to show the conclusions of a number of experimenters regarding the value of K for 90° bends in closed conduits. The result shows an extreme lack of agreement, which leads to the conclusion that the loss is not independent of the kind of pipe used in the test. The following discussion would seem to substantiate this conclusion.

The loss of head in bends in excess of that in a straight pipe of equal length, is probably caused by two predominating conditions: namely, increased skin friction and eddy losses.

Investigations seem to indicate that, owing to centrifugal force, the flow must adjust itself in the bend to a greater ratio of maximum to average velocity. As skin friction varies approximately as the square of the velocity, the skin friction would naturally be greater than in a straight conduit, and hence this loss is dependent upon the length of the bend and the type of the conduit.

Eddy losses are caused by an adjustment of the stream lines at the entrance and at the exit of the bend. These losses are probably independent of the length of the bend.

Recent experiments for closed conduits have tended to show that bends having a ratio of radius of bend to diameter of conduit equal to about 5 correspond to the minimum loss in the bend, and hence to the minimum value of K as indicated in Fig. 66.

This bend ratio of 5 may be the critical value, where the eddies caused by the adjustment of stream lines at the entrance to the bend are somewhat smoothed out by overlapping the eddies caused by the adjustment of stream lines at the exit of the bend. In other words, it is possible to conceive of a condition where the eddies at entrance and exit tend to synchronize or partially counteract each other for a bend ratio of a certain value. Although experimental data are meager, there seems to be no good reason to conclude that a bend ratio of about 5 will give greater losses than bend ratios in excess of that value.

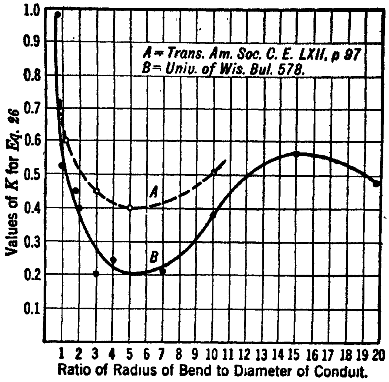


FIG. 66.

Figure 67 gives recommended values of K for 90° bends in closed conduits, based on available data. The curve representing safe values is an enveloping curve for all experiments, and the curve representing probable values is the mean of some of the more recent experiments. Fifty per cent should be added to the values of K from Fig. 67 for screwed pipe elbows on account of the sudden enlargement and contraction in such fittings.

The two conditions previously described, governing the losses in bends of

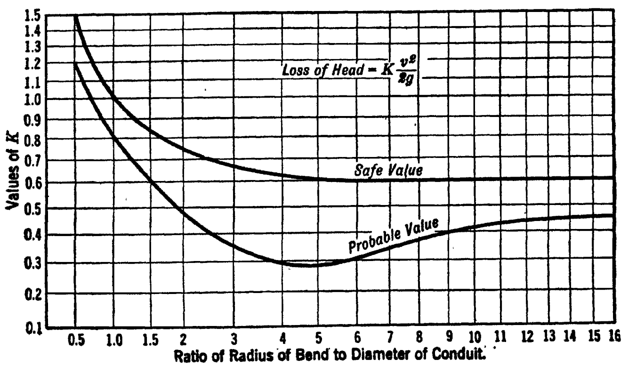


FIG. 67.—Recommended Values of K for 90° Bends in Closed Conduits.

closed conduits, seem to indicate that the loss due to skin friction varies approximately directly as the angle of the bend (length of curved part for same radius); but the losses at entrance and exit are somewhat independent of the angle of the bend. It would therefore seem logical to assume that the loss in a 45° bend would in general be greater than one-half the loss in a 90° bend. While no data are available upon which to base reliable conclusions,

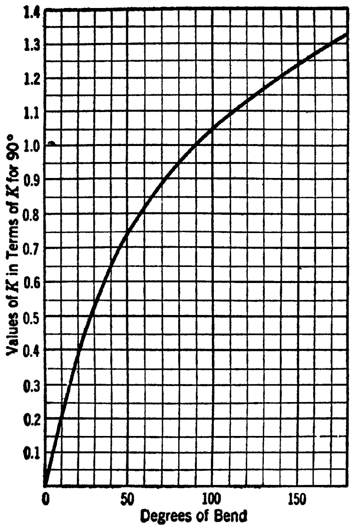


FIG. 68.—Values of K for Various Degree Bends in Terms of K for 90° Bends.

it is recommended that values of K for bends other than 90° be taken from Fig. 68.

Few experiments have been made on the losses in bends of open conduits. Fortunately, sharp bends in open conduits are not usually required. Writers generally allow for loss at bends of open conduits by adjusting the coefficient, n, of the flow equation.[5] Frequently a coefficient is given for a straight open conduit and another coefficient for the same type of conduit "with moderate curvature" or other equally indefinite description. The author claims that an allowance for bends should not be made in the flow coefficient, n; but that the slope for straight alinement should be determined first, and increased slopes provided at bends corresponding to the best judgment regarding the additional losses at the bends. The few experiments that have been made seem to indicate that the loss due to bends in open conduits is much less than that for closed conduits, and values equal to one-half of those in Fig. 67 are recommended. Such values have been compared by the author with experimental data and have been found to agree very closely when one-half the "probable curve" value is used.

For reverse bends in both open and closed conduits, the loss is probably somewhat greater than if the two bends were separated by a considerable length of straight pipe, but the amount of such loss has not been determined.

Recommended safe values of K for miscellaneous fittings are given in Fig. 69.

71. Losses at Conduit Valves.—The value of K for use in Eq. (26) for wide-open gate valves is probably less than 0.1 although few experiments are available.

E. A. Dow's experiments on the disk-arm type of butterfly valves (Figs. 261 and 262) indicate a value of K equal to:

$$K = \frac{t}{d}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (29)$$

where t = the thickness of the disk of the valve, and
d = the diameter of the valve.

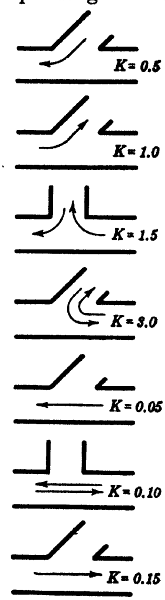


FIG. 69.—Recommended Safe Values of K for Miscellaneous Fittings.

[5] Mention is made of this in Sec. 73 but it is not recommended.

The value of v for use in Eq. (26) is that in the normal section of the conduit.

The writer has been able to find only three experiments on valves of the plunger type (Figs. 264 and 265). On the basis of these experiments, he has devised the following equation:

$$K = \frac{0.183}{\sqrt[3]{d}}. \qquad (30)$$

Where d is the diameter at the small end, in feet. The value of v for use in Eq. (26) is that in the small end of the valve. Results from Eq. (30), compared with experimental data, are as follows:

Source	d	v	Experimental Feet Loss	Experimental K	K from Eq. (30)
Larner Eng. Co.	1.5	20.0	1.00	0.16	0.16
Wellman, Seaver, Morgan Co.	4.67	19.0	0.65	0.116	0.11
Larner Eng. Co.	12.0	20.0	0.50	0.08	0.08

72. Miscellaneous Conduit Losses.—As all losses other than skin friction and bends are due to sudden changes in section of the conduit, a knowledge of the laws governing the losses due to sudden contractions and enlargements will assist materially in determining the losses in various irregularities of conduits.

The theoretical value of K (for use in Eq. (26)) for a sudden enlargement in section of a closed conduit is:

$$K = \left(1 - \frac{a_1}{a_2}\right)^2, \qquad (31)$$

where a_1 and a_2 are the areas of the smaller and larger sections, respectively. The value of v for use in Eq. (26) is that in the smaller section. Actual losses follow the theoretical losses very closely. Losses due to gradual enlargements in a closed conduit are not known exactly. Etcheverry gives the value of K as follows:

$$K = \left(1 - \frac{a_1}{a_2}\right)^2 \sin\theta, \qquad (32)$$

where θ is the central angle formed by the taper.

Equations (31) and (32) are frequently used to determine the value of K for sudden enlargements in open conduits and for the junction of a closed and open conduit; but this is not correct, as the fundamental principles used in their derivation do not apply to open-water conditions. If the maximum possible loss is to be determined, the value of K should be taken from the following equation:[6]

$$K = 1 - \frac{a_1^2}{a_2^2}. \qquad (33)$$

The value of v for use in Eq. (26) is that in the smaller section.

[6] The exception is where the hydraulic jump is introduced.

This loss is reduced for gradual enlargements in open conduits. Unfortunately, the ratio between the rate of enlargement and the loss is not known exactly; but the loss may not exceed 25 per cent, i.e., $K = 0.25$, if the angle of flare of each side of the conduit makes an angle with the center line not in excess of 2 or 3 degrees.

For sudden contractions in closed conduits, the values of K are given in Fig. 70, which was derived in a manner similar to that employed by Merriman, except that the coefficient of contraction for a sharp-edged orifice, equal to 0.60, was used instead of 0.62 as employed by him in his basic equation. The value of v for use in Eq. (26) is that in the smaller section. Losses due to gradual contractions are very small.

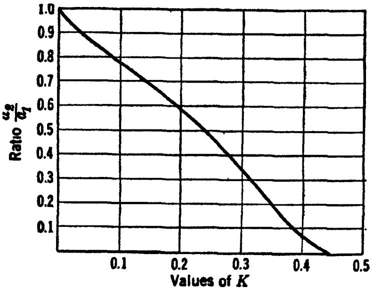


FIG. 70.—Values of K for Sudden Contractions in Closed Conduits.

The loss of head in sudden contractions in open conduits is covered in the explanation of the application of Eq. (43) of Sec. 74 to the discharge through restricted channels. The loss in gradual contractions of open conduits is negligible.

73. Skin-friction Conduit Losses.—Most of the many empirical equations for the uniform flow of water in conduits are based on the following expression, developed by Chezy in 1775:

$$v = CR^{1/2}S^{1/2}, \text{ or in its more general form,}$$

$$v = CR^nS^m, \qquad (34)$$

where v = the average velocity in feet per second;
R = the hydraulic mean radius in feet, being the cross-sectional area divided by the wetted perimeter;
S = the sine of the slope of the hydraulic gradient [7] or water surface;
C, n and m = coefficients depending upon the type of the conduit and the condition of the wetted perimeter, all of which must be determined by experiments.

None of the empirical equations agree very closely with one another and, moreover, they all embody the coefficient C, which must be adopted according to judgment as indicated by available experiments, depending upon the nature and condition of the wetted perimeter. In some equations, n and m are constant for all types of conduits. Other writers have suggested different values for each type of conduit.

A sufficient number of experiments have not been made to enable the engineer to adopt precise coefficients, C. The character of the wetted perimeter

[7] For a definition of "hydraulic gradient," see Sec. 76.

is liable to considerable change during the life of the conduit, due to corrosion, tuberculation, growth of fungi, weeds, and other vegetation, silt deposits, scour, ice, and other things. The chief difficulty in the use of existing experimental data is the lack of the power of definite description of the condition of the wetted perimeter of the experimental conduit.

For these reasons, a great refinement in flow equations seems unwarranted at the present time, and a considerable factor of safety must be adopted to compensate for possible errors in the adopted equation and in the choice of the coefficient C.

Up to the present time, most experiments on flow in open conduits have been published in terms of Kutter's equation, and most of those on flow in closed conduits in terms of Hazen and Williams' equation. These equations will be used exclusively herein.

Kutter's equation is:

$$v = \left\{ \frac{\frac{1.811}{n} + 41.66 + \frac{0.00281}{S}}{1 + \left[41.66 + \frac{0.00281}{S}\right]\frac{n}{\sqrt{R}}} \right\} \sqrt{RS}. \quad . \quad . \quad . \quad . \qquad (35)$$

Hazen and Williams' equation, as published in their "Hydraulic Tables,"[8] is:

$$v = CR^{0.63}S^{0.54}0.001^{-0.04}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (36)$$

and this may be reduced to:

$$v = 1.32CR^{0.63}S^{0.54}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (37)$$

where v, R and S = the values given for Eq. (34), and
C and n = coefficients which depend upon the nature and condition of the conduit and which must be determined experimentally.

The diagram, Fig. 71, for the solution of problems by Kutter's formula covers a range far beyond anything hitherto offered in a single sheet. In essentials the diagram consists of two parts, the upper part giving simultaneous values of R and n and the lower part giving simultaneous values of S and V. The two parts are connected by a series of guide lines. The solution to a single problem is given by the dotted lines. For a value of R = 2.6 ft. and of n = 0.015 and a slope of 0.00125, the indicated V = 6.7 ft. per second. (The actual numerical answer is 6.66 ft. per second.)

The construction of the diagram is based on the fact that for all slopes and their corresponding velocities, considered for one value of R and one value of n, there will be another value of R to satisfy another value of n. For example: Note that the same guide line practically goes through the intersection of R = 0.2 and n = 0.010 and also through the intersection of R = 2.0 and

[8] John Wiley & Sons, 1920.

$n = 0.045$. Reference to standard tables for Kutter's formula shows that the velocities for any given slope are practically identical for these corresponding related values of R and n. If curved lines are used the plat is readily constructed, but other conditions must be considered to make all lines on the diagram straight. For slopes flatter than about 0.0003, the above argument is not strictly true. This is indicated by the fact that any given guide line splits into diverging curves. For the heavy guide lines the right-hand curves are lettered to indicate their use for values of $n = 0.030$ and the left-hand curves for $n = 0.012$. Of course, the right and left curves from the lighter guide lines are for the same respective values. Interpolation between the proper guides can be estimated for other values of n.

For steep slopes such as are found on many of the western irrigation systems in long chute-drops and in spillway flumes from power penstock surge tanks, the area of the water-and-air prism can be found by using about the same values of n as for ordinary slopes. This holds true at least up to velocities around 40 ft. per second. The diagram can still be used even though the intersection of S and V must be projected beyond the values as drawn and the intersection comes within the part of the diagram showing simultaneous values of R and n. Values from the diagram should not be given to more than two significant figures. This will be closer than necessary assumptions will warrant.[9]

Figure 72 is a diagram for the solution of Hazen and Williams' equation. Given $v = 11.0$ ft. per second and $C = 110$, find $v/C = 0.1$ at left-hand margin. Trace horizontally to conduit diameter or hydraulic radius (say 10.0 ft. diameter) and find loss of head at bottom margin equal to 2.9 ft. per 1000 linear ft. of conduit.

As the coefficients n and C are as yet known only approximately, it is necessary to adopt such values as will result in an error, if any, on the side of safety. If, in a pipe line, the head lost in friction is only a small part of the total head available for power, it is customary when estimating power available to determine friction head by the selection of a probable coefficient. On the other hand, the design of a surge tank demands the selection of a coefficient corresponding to the least possible friction when the surge tank is filling and a coefficient corresponding to the greatest possible friction when the tank is emptying. This results in the greatest possible surges in the tank.

In the case of an open conduit, an error in the selection of the coefficient has considerable effect on the capacity and, if a probable coefficient is used to determine the depth of water for a given discharge, a free-board or super-elevation of the sides of the conduit above calculated water surface is provided to insure a margin of safety and to provide space for wave crests. This free-board should be of such height that, with a coefficient corresponding to maximum possible friction, the conduit will accommodate the depth required for the given discharge.

[9] This diagram was especially drafted by Fred C. Scobey for this book. He offered this form of diagram for the first time in "The Flow of Water in Irrigation Channels" (Bul. 194, U. S. Dept. of Agr., 1914). Continued investigations of the Division of Agricultural Engineering enabled him to perfect this diagram to the form herein given.

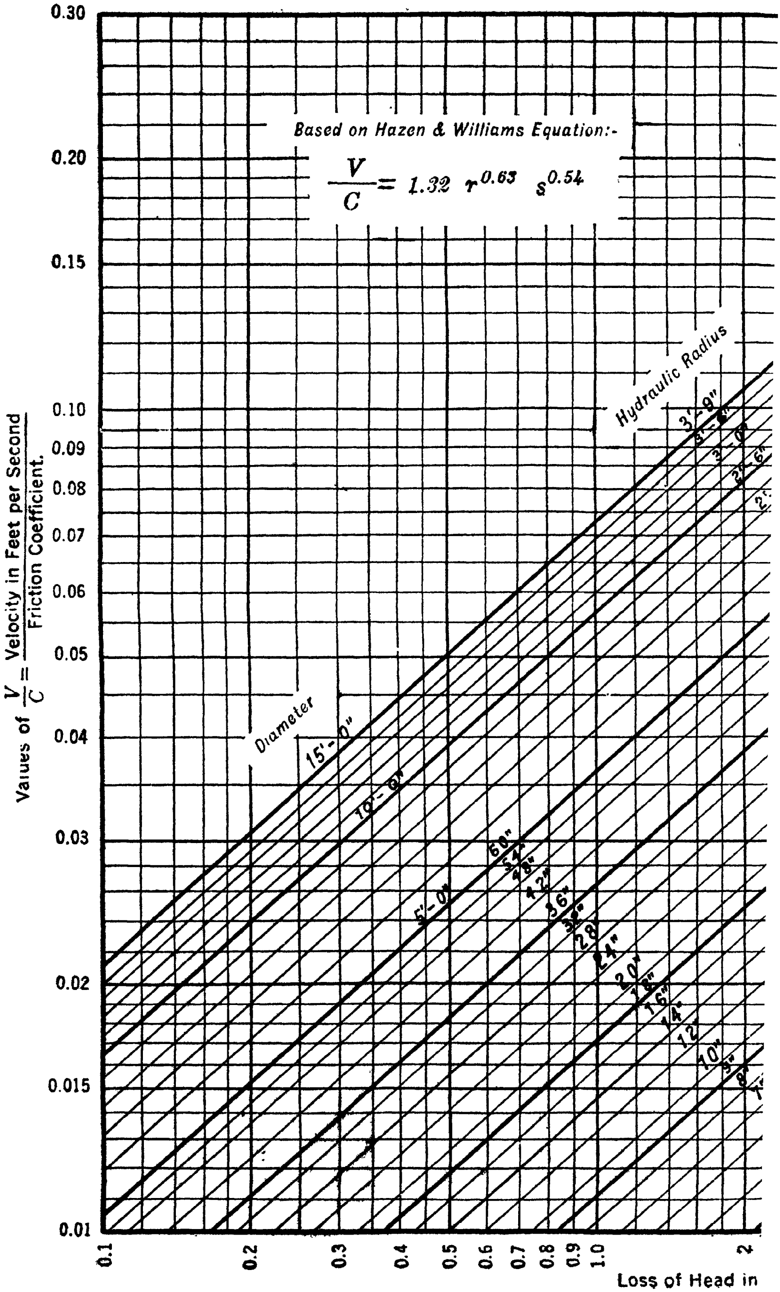
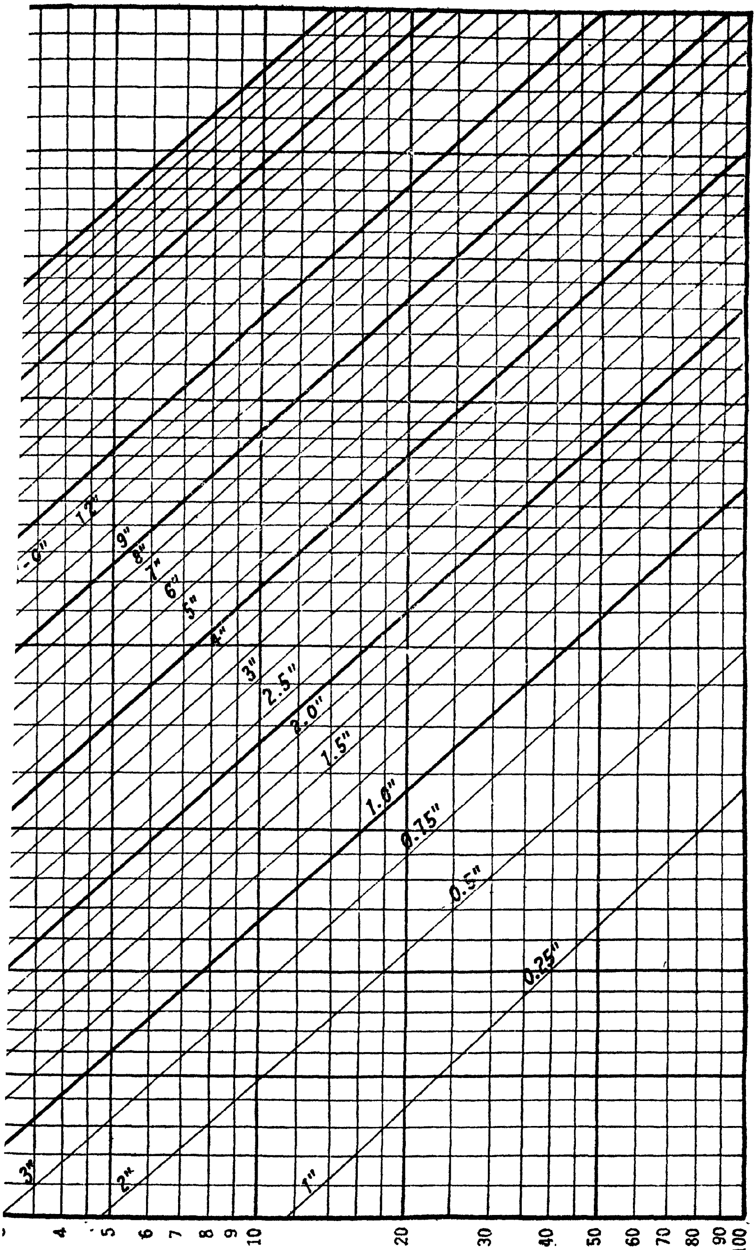

Based on Hazen & Williams Equation:-
V/C = 1.32 r^0.63 s^0.54
Values of V/C = Velocity in Feet per Second / Friction Coefficient.
0.30
0.20
0.15
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.015
0.01
Hydraulic Radius
3'-9"
3'-0"
2'-6"
Diameter
15'-0"
10'-0"
5'-0"
60"
54"
48"
42"
36"
32"
28"
24"
20"
18"
16"
14"
12"
10"
9"
8"
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
2
Loss of Head in

FIG. 72.—Fric



r 1000 Linear Feet of Pipe

s Diagram for Closed Conduits.

To face page 124

Many other cases may be cited where the minimum, maximum, or probable capacity of the conduit must be estimated. The tabulations given in Tables XXII and XXIII have been made from a study of all available sources of information regarding experimental determination of the friction coefficient.[10] The values corresponding to best and worst conditions embrace practically all variations in the data studied. Isolated values, outside the limits given, have been excluded where there was reasonable doubt as to their accuracy. It is felt that the values given cover the range of practical considerations. The *probable* values are those ordinarily used in practice.

Unless otherwise stated, the friction coefficients in the tables are based on straight or slightly sinuous conduits free from the following influences:

(*a*) Curvature, other than slightly sinuous;
(*b*) Settlement of open conduits or other defects of construction;
(*c*) Sediment, rocks, or other deposits washed or fallen in;
(*d*) Plant growths; moss, etc.;
(*e*) Ice covering;
(*f*) Wind movement.

All experimental data were adjusted as closely as possible to correct for curvature so that all coefficients are practically for straight conduits. A correction to the calculated slope should be made for appreciable curvature, as indicated in Sec. 70.

Obviously, the results of defects in construction, settlement, and obstructions are indeterminate although an attempt has been made in the tables to give approximate coefficients for certain conduits which should cover the range of reasonable maintenance.

It is not known that any experiments have been made to determine the friction coefficient of the under side of a sheet of ice. If the ice had the same effect on friction as the bottom and sides of the conduit, the determination of velocity would be made in the same way as for open-water conditions except that the width of the ice sheet would be included in the wetted perimeter and the area would be the area under the ice. Although experimental data are not available, it is known that the retarding effect of the under side of an ice sheet is much less than that of the ordinary form of earth and rock canals and probably equal to that of the smoothest conduits. In the absence of a better method, it is recommended that, for ice conditions, the coefficient for open-water conditions be used, and a percentage of the width of the ice sheet, depending upon the roughness of the conduit, as indicated below, be included in the wetted perimeter.

[10] Compiled by F. C. Scobey, M. Am. Soc., C. E. Mr. Scobey has had charge of the experiments and compilation of data on the carrying capacity of water conduits for the U. S. Department of Agriculture, Division of Agricultural Engineering, for the past twelve years, and is author of the following publications on this subject. These may be obtained on request from the Department of Agriculture at Washington, D. C.

U. S. Dept. Agr. Bul. 194. The Flow of Water in Irrigation Channels.
U. S. Dept. Agr. Bul. 376. The Flow of Water in Wood Stave Pipe.
U. S. Dept. Agr., Bul. 852. The Flow of Water in Concrete Pipe.

Value of the Coefficient for Conduit under Open-water Conditions	Percentage of the Width of Ice Sheet to be Included in Wetted Perimeter
0.010	100
0.015	87
0.020	76
0.025	66
0.030	58
0.035	50
0.040	43
0.045	37
0.050	32
0.055	28
0.060	25

The probable maximum thickness of the ice sheet depends upon climatic conditions at the site, the length, width, and depth of the conduit, and the velocity, and can be estimated only by comparison with similar conduits in the vicinity.

TABLE XXII

RECOMMENDED VALUES OF n FOR REASONABLY STRAIGHT OPEN CHANNELS

Dry Excavated Earth Canals:

Generally speaking, the value of n in earth canals increases with the life of the canal unless constantly maintained. Slightly silted waters will "slick" over an original rough surface so that the value of n becomes less. Heavily silted water will make n less, but it will also decrease the area of the water prism.

Best conditions are found in tough silt or clay soils, with velocities below scouring limits. $n = 0.016$ may be acquired by silt deposit free from growths.

New canals in sandy loam to clay loam range from class above to one next below.. $n = 0.020$

The accepted values for medium to large canals in firm earth or gravelly loam with silty water, operated by organization that will give reasonable maintenance.. $n = 0.0225$

Small ditches, easily influenced by slight roughness, and larger canals poorly maintained... $n = 0.025$

Mountain power canals with cobble bottoms but without finer materials for a graded bedding... $n = 0.028$

Dredged Earth Canals:

A dredged channel is rougher than one excavated by hand and teams. Likewise, a dipper dredge leaves a rougher bottom than does a drag line. Differences in value of n are largely brought about by adaptability of soil types and silt in the water to smooth over the original roughness.

Best conditions.. $n = 0.0225$
Average conditions... $n = 0.030$
Worst conditions without neglect of maintenance................. $n = 0.040$

Canals Excavated in Rock:

It is possible for rock excavation to be done in horizontally stratified rock, resulting in a very smooth bottom. Such canals, if very wide, will have a very low value of n as the rough sides have relatively little influence and, if the canal is of ordinary size and no attempt is made to smooth up the sides materially, it is considered that the minimum value of n is about.... $n = 0.020$

Usual or average conditions, with care in smoothing the rock cut by breaking off projections.. $n = 0.033$

Worst possible conditions—there is no limit but it is seldom above...... $n = 0.045$

Silt and gravel deposits in rock canals may lower n by filling in the holes in the bottom.

TABLE XXII.—*Continued*

Natural Channels:

It is impossible to describe accurately the conditions of a natural channel that correspond to any given value of n.

The best natural channels have a value of n seldom below............ $n = 0.025$

Average natural channels have a value of n probably in the neighborhood of.. $n = 0.030$

Worst possible conditions—No limit.

Judgment and experience are necessary to fix the value of n accurately for natural channels.

Concrete Linings:

The value of n depends upon the specifications for the concrete surface and the workmanship. Considerable variation in n may result under the same specifications but with different workmen. It should be borne in mind that more favorable values may be attained in construction than should be anticipated in design. The following are the general characteristics:

Best possible, with neat cement, extremely well troweled surface. Seldom realized in practical construction............................ $n = 0.010$

The highest grade of practical concrete linings in best condition. Surface troweled as smooth as hand troweled sidewalks. Expansion joints perfectly smooth.

Best.. $n = 0.011$
Probable.. $n = 0.012$
Worst... $n = 0.013$

Surface as left by smooth jointed forms, or roughly troweled. Expansion joints fair. The probable value is usually adopted for concrete linings:

Best.. $n = 0.013$
Probable.. $n = 0.014$
Worst... $n = 0.015$

Concrete having prominent form marks, or previous types subject to deposits of stones on the bottom:

Best.. $n = 0.015$
Probable.. $n = 0.016$

Worst or probable maximum value not subject to rejection because of bad workmanship.. $n = 0.018$

If liable to a growth of moss, the foregoing values should be increased by adding... 0.002

Values of n in excess of 0.017 indicate very poor concrete, which may be either badly spalled, owing to frost or great difference in mixtures of concrete and finish coat, or broken down by the action of alkali. Similar values hold where the channel is losing its identity as a concrete surface, by deposits of sand and gravel or by moss or larval growths. Both moss and larvae appear to thrive in high velocities, even those of 30 or 40 ft. per second. Covering a channel to exclude sunlight is effective in preventing moss growths and would also tend to prevent larval growth, which is so well known in the Pacific Northwest.

Gunnite Linings:

Concrete linings deposited by a cement gun, from the inside:
Best, if scrubbed with wire brush.................................. $n = 0.016$
Average, if not scrubbed... $n = 0.019$
Worst, for poor workmanship.. $n = 0.021$

Following a cement gun with a trowel will improve the surface from a capacity standpoint but may induce seepage loss by impairing the original density attained by the process. It is, however, recommended that the surface "rebound" be scrubbed off with a wire broom before it hardens and sticks to the canal bed.

TABLE XXII.—*Continued*

Miscellaneous Masonry Linings:

Glazed brickwork:

Best	$n = 0.011$
Probable	$n = 0.013$
Worst	$n = 0.015$

Brick in cement mortar:

Best	$n = 0.012$
Probable	$n = 0.015$
Worst	$n = 0.017$

Dressed ashlar surface:

Best	$n = 0.013$
Probable	$n = 0.015$
Worst	$n = 0.017$

For bench flume, consisting of natural rock surface for the uphill side, a smooth concrete retaining wall on the downhill side, and with a floor between. Floor lined with concrete and clean. Uphill side without projecting points .... $n = 0.020$

Same as above; but floor covered with sand or gravel, or left as excavated without projections, uphill side with a few projecting points, such as obtained with careful excavation in hard rock .... $n = 0.025$

Cement—rubble surface:

Best	$n = 0.017$
Probable	$n = 0.025$
Worst	$n = 0.030$

Dry-rubble surface:

Best	$n = 0.025$
Probable	$n = 0.033$
Worst	$n = 0.035$

Wooden Box Flumes:

The following applies to well-constructed flumes, carefully maintained, without projecting calking or other imperfections. Battens where used to be included in wetted perimeter:

Planed lumber, longitudinal boards sides and bottom:

Best	$n = 0.011$
Probable	$n = 0.014$
Worst, after years of service	$n = 0.018$

Unsurfaced lumber, longitudinal boards sides and bottom:

Best	$n = 0.012$
Probable	$n = 0.015$
Worst, after years of service	$n = 0.018$

Roofing paper lining varies with the type generally from.. $n = 0.010$ to $n = 0.017$

In general, the best and worst values correspond to new and very old flumes respectively, the latter with patches here and there in place of complete renewal of rotted members and corresponding to rather faulty work.

Wood-stave flumes:

Creosoted:

Best	$n = 0.011$
Probable	$n = 0.012$
Worst	$n = 0.014$

TABLE XXII.—*Continued*

Untreated:

Best .... $n = 0.010$
Probable .... $n = 0.0115$
Worst .... $n = 0.014$

Smooth-Interior Steel Flumes:

For smooth-interior flumes, as manufactured and erected under various trade names:

When unpainted:

Best condition .... $n = 0.0105$
Probable condition .... $n = 0.012$
Worst condition .... $n = 0.014$

When painted:

Best condition .... $n = 0.012$
Probable condition .... $n = 0.013$
Worst condition .... $n = 0.017$

The condition of these patent-joint flumes is largely a function of size and the number of carrying rods. Small flumes (2 to 5 ft. in diameter) maintain their catenary shape quite well, if carrying rods are installed midway of each sheet, as well as at the ends, or if the side girders are set close to the sheets. Either method prevents excessive "scalloping." Large flumes require very frequent carrying rods and heavy gage metal.

Rough Interior Steel Flumes:

Many of the older installations in this country include steel flumes with projecting band joints or steel flumes of corrugated sheet metal. Both of these types are now out of date and will not be discussed further than this: A fair value of n for the first type was 0.017, and for the second type 0.022.

TABLE XXIII

RECOMMENDED VALUES OF HAZEN AND WILLIAMS' C FOR STRAIGHT CONDUITS

Cast-Iron Pipe:

On account of the growth of tubercles on the inside of the pipe, which decreases its area as well as increases its roughness, the value of C for a given age decreases as the diameter, but variation of C with age depends largely upon the composition of the water flowing in the pipe. Values are, therefore, rough approximations.

Hazen and Williams recommend the following average values of C:

Diameter of Pipe in Inches	Age In Years						
	0	5	10	20	30	40	50
4	130	118	107	89	75	64	55
8	130	119	109	93	83	73	65
12	130	120	111	96	86	77	70
16	130	120	112	98	87	80	72
24	130	120	113	100	89	81	74
30	130	120	113	100	90	83	76
36	130	120	113	100	90	83	76
40	130	120	113	100	90	83	77
60	130	120	113	100	90	83	77

TABLE XXIII.—*Continued.*

Cleaning old pipe increases the coefficient materially. Experimental values of $C = 150$ have been obtained for best new cast-iron pipe.

Riveted Steel Pipe:

According to Hazen and Williams, the average value of C for light-riveted steel pipe of any age is the same as that for cast-iron pipe ten years older.

Experiments seem to indicate, however, that for very heavily riveted steel pipe the average value of C for any age is the same as that for cast-iron pipe twenty years older. Experiments are meager. The same uncertainties exist as in the case of cast-iron pipe.

Experimental values of $C = 140$ have been obtained for light-riveted best new steel pipe.

Welded Steel Pipe:

Sufficient experiments are lacking. The same uncertainties exist as in the case of cast-iron pipe. Average values of C for any age may be taken as the same as those for cast-iron pipe five years older.

Wood-stave Pipe:

Best information seems to indicate that C does not vary materially with age for wood-stave pipe. Experimental values differ materially, ranging as follows:

Best	$C = 145$
Probable average	$C = 120$
Worst	$C = 110$

Unlined Tunnels in Rock:

No experiments, based on other equations than that of Kutter, have been made. Unless more than usual information as to detail of the surface can be anticipated, use value of

$$n = 0.035 \text{ to } n = 0.040$$

depending on importance of definite capacity, and compute with the theoretical neat section. In construction, overbreak will run from 10 per cent to 30 per cent, making a larger perimeter and greater area than the theoretical one but giving a correspondingly lower velocity. Two sets of tests disclosed values of n slightly less than 0.035, using a measured discharge but a theoretical neat-section hydraulic radius.

Concrete Pipes and Concrete-lined Pressure Tunnels:

In all equating of performance, a definite class of concrete must be considered. Our smoothest water conveyors are of concrete; likewise the roughest ones are of concrete. Great capacity is attained by a few simple specifications:

(1) Use fairly rich, dense, well-worked concrete.
(2) Use forms that are sturdy enough to retain shape under pressures of wet concrete and jarring of necessary form tapping.
(3) Use heavy oiled metal sheets for surface in contact with concrete.
(4) Use enough forms to insure rigid set before removal of forms.
(5) Omit a wash coat (Density prevents percolation).

All first-cost expense, necessary to first-class work, will yield tremendous dividends in the saving of lost head.

For pipes large enough to permit men to work inside them, for large cement-lined steel pipe, and for concrete-lined pressure tunnels built on above specifications:

Best workmanship	$C = 150$
Probable workmanship	$C = 140$
Worst workmanship	$C = 120$

If the above specifications are not rigidly adhered to a sharp decline in capacity results:

Wood forms; cracks between boards; slight settlement and squeezing of forms; slight offsets between joints of pipe or set-ups of monolithic work:

Best workmanship	$C = 130$
Probable workmanship	$C = 120$
Worst workmanship	$C = 90$ or less

TABLE XXIII.—*Continued*

Where something rougher than the expected surface is disclosed on the removal of forms, then the value of C may be raised from 10 to 20 points by chipping down roughness, tapering offsets, filling cavities, and smoothing on a rich grout.

Slimes and moss growths come quickly or not at all. If they are anticipated, allow for them in design, as they rarely grow beyond a stage which is quickly attained; therefore, periodic cleaning, in order to be of benefit, would have to occur oftener than is usually practicable.

While slimes generally result in less pipe capacity, the thinnest of slick slimes apparently yields an increase. However, if slimes are expected, it would be well to allow for a decrease in capacity and the following should be used:

20 points below those given for best workmanship,
15 points below those given for average workmanship, and
10 points below those given for worst workmanship.

74. Flow over Dams.—The basic theoretical expression for the flow over weirs is given in Eq. (24) of Sec. 64, which, if all constants are combined, may be written:

$$Q = Clh^{3/2}, \qquad (38)$$

where Q = the total discharge, in cubic feet per second;
C = the coefficient of discharge, which depends on the shape of the crest and the head on the crest;
l = the net or effective length of crest, i.e., the total length of crest corrected for end contractions due to piers and sharp-cornered abutments;
h = the actual or measured head on the crest, taken at a point sufficiently remote from the dam to avoid the surface curve.

Francis has determined that, to allow for the effect of the velocity of approach, this equation should be written:

$$Q = Cl\{(h + h_v)^{3/2} - h_v^{3/2}\}, \qquad (39)$$

where h_v = the head corresponding to the velocity of approach. An approximate form of Francis' equation is:

$$Q = Cl(h + h_v)^{3/2}. \qquad (40)$$

Eq. (40) gives values of Q in excess of that from Eq. (39) as indicated in Fig. 73.

It will be noted that the error for a depth of channel of approach greater than twice the head on the crest is less than 1 per cent.

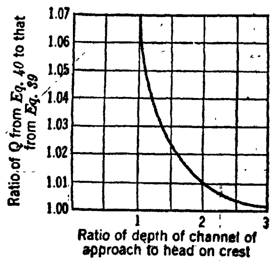


FIG. 73.

Francis' equation for the necessary correction due to complete end contractions is:

$$l = l_t - 0.1nh, \qquad (41)$$

where l_t = the total or gross length of crest between abutments and piers (Fig. 74), and
n = the number of complete end contractions.

If the crest is obstructed by piers having considerable widths and sharp corners, as indicated in Fig. 74, n represents the number of corners that serve

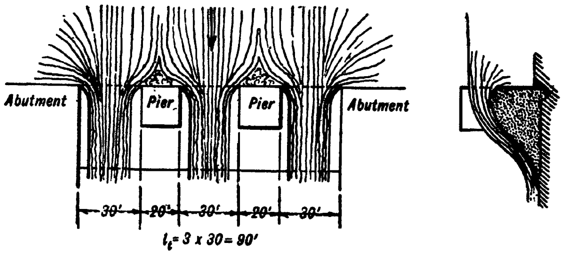


FIG. 74.—Complete Contractions Due to Abutments and Large Piers.

to deflect the water, there being six complete contractions in this instance, two for each pier and one for each abutment. Usually, however, the piers are relatively thin and are provided with sharp upstream ends, as indicated in

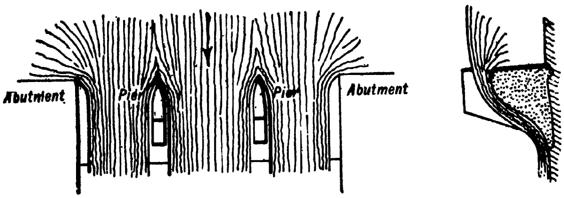


FIG. 75.—Partial Contractions Due to Sharp Piers.

Fig. 75. In such cases the contractions for the piers are not complete, and Francis' equation would give too small values of l. The number of experiments has not been sufficient to determine closely the effect of piers on the effective length of crest.

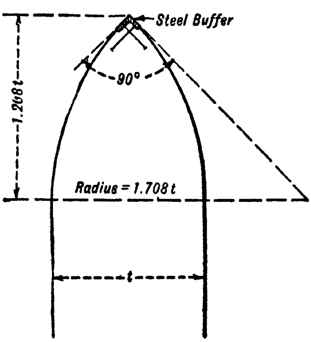


FIG. 76.—Typical Sharp Nose Pier.

Unless the piers are unusually thin, relative to the head on the crest, or very considerably pointed upstream, each may be considered to offer two partial contractions, probably amounting to not more than $0.04h$ for each contraction, in case the piers are pointed as indicated in Fig. 76, and varying between this and $0.1h$ for thick, blunt piers, depending on the degree of sharpness and the relative thickness. Abutments having corners, as indicated in Fig. 62, or abutments extending for a considerable distance upstream offer no contraction to the crest length. The contraction for each abutment varies, therefore, between zero and $0.1h$, the latter being for sharp-cornered abutments as shown in Fig. 75.

Francis' equation may then be written:

$$l = l_t - h(C_a n_a + C_b n_b \ldots C_n n_n), \quad \ldots \ldots \quad (42)$$

where, C_a, C_b, etc., represent the contraction coefficients applicable to the several different contractions which may be expected, and

n_a, n_b, etc., the number of contractions having contraction coefficients, C_a, C_b, etc., respectively.

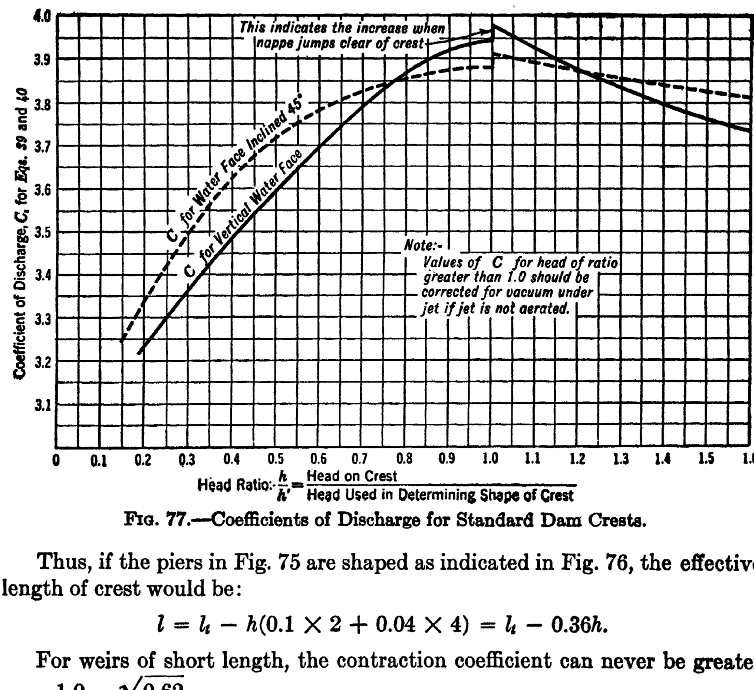


FIG. 77.—Coefficients of Discharge for Standard Dam Crests.

Thus, if the piers in Fig. 75 are shaped as indicated in Fig. 76, the effective length of crest would be:

$$l = l_t - h(0.1 \times 2 + 0.04 \times 4) = l_t - 0.36h.$$

For weirs of short length, the contraction coefficient can never be greater than $\frac{1.0 - \sqrt{0.62}}{2} = 0.106$ times the clear distance between two piers or abutments, as this corresponds to the side contraction through sharp-cornered orifices having a contraction coefficient of 0.62. Therefore, the contraction coefficient is 0.106 for all cases where the depth is greater than 1.06 times the length of spillway.

The value of the discharge coefficient, C, for use in Eqs. (39) and (40) has been determined experimentally for spillways of many different types. These experiments have been carefully tabulated in Water Supply Paper, 200.[11] Unfortunately, no experiments have been made for the modern type of standard dam crests described in Sec. 103. Values of C applicable to the discharge

[11] "Weir Experiments, Coefficients, and Formulas," by R. E. Horton, U. S. Geol. Survey Water Supply Paper, 200, 1907.

over such crests, derived by the author from theoretical considerations and a comparison of a number of experiments on similar shapes of crests, may be taken from Fig. 77.

The standard dam crest is designed practically to fit the shape of the nappe corresponding to the maximum flood to be expected. For low heads the coefficient approximates that for broad-crested weirs, and, for heads, h, greater than the head, h', for which the crest is designed, the jet jumps free as indicated in Fig. 77 by the break in the curves.

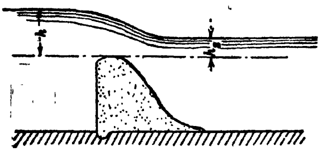


FIG 78.—Submerged Crest.

Submerged Spillways.—If the crest of the spillway is submerged, as in Fig. 78, the discharge coefficients, for use in Eqs. (39) and (40) should be modified according to the degree of submergence, as indicated in Table XXIV.[12] In this table, C is the coefficient for *free* discharge over a similar crest under the same head, and C' is the modified coefficient due to the submergence. The heads are h and h_s as in Fig. 76.

TABLE XXIV

RELATIVE COEFFICIENTS, SUBMERGED CREST AND FREE CREST

$\frac{h_s}{h}$	$\frac{C'}{C}$	$\frac{h_s}{h}$	$\frac{C'}{C}$
0.0	1.000	0.5	0.937
0.1	0.991	0.6	0.907
0.2	0.983	0.7	0.856
0.3	0.972	0.8	0.778
0.4	0.956	0.9	0.621
...	.....	1.0	0.000

If a standing wave occurs below the crest, as indicated in Fig. 79, the effect of submergence is lost and the discharge is the same as for a free crest.

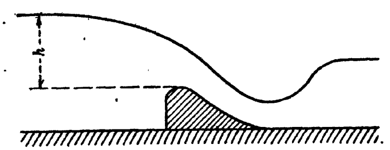


FIG. 79.—Submerged Crest with Standing Wave.

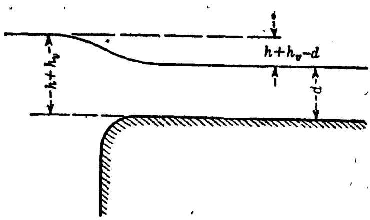


FIG. 80.

Broad-crested Spillways.—Fig. 80 indicates a theoretical broad-crested weir having:

[12] From U. S. Deep Waterways Experiments. See U. S. Water Supply Paper No. 200, p. 146. These experiments were made on a model having a rounded crest, approximating more closely than any of the others to the shape of a standard dam crest.

(1) No surface friction;
(2) No crest contraction;
(3) No end contractions.

Therefore, there are, for this theoretical case, no eddy or other friction losses.

The velocity of the water on the crest is due to the head $(h + h_v) - d$ and is equal to:

$$v = \sqrt{2g(h + h_v - d)}.$$

The discharge is therefore equal to:

$$Q = Av = ld\sqrt{2g(h + h_v - d)}. \quad . \quad . \quad . \quad . \quad . \quad (43)$$

For this case the water will discharge in accordance with the principle of least energy, which obtains when the value of d corresponds to the maximum flow. This value [13] of d is:

$$d = \frac{2(h - h_v)}{3},$$

which, substituted in Eq. (43), gives,

$$Q = 3.087l(h + h_v)^{3/2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (44)$$

Equation (44) is applicable to types of spillways indicated in Fig. 81, commonly used as an adjunct to earth dams. In such cases the crest of the spillway must be sloped to an extent sufficient to carry away the discharge at depth $d = \frac{2(h + h_v)}{3}$, which is the equivalent of the condition of frictionless surface in the theoretical case. The crest of such spillways is usually well rounded to obviate crest contraction. In case there are end contractions, the value of l given in Eq. (42) should be used. If h is measured to a large pond, then h_v is neglected, but there should be subtracted from h the friction loss between the pond and the crest.

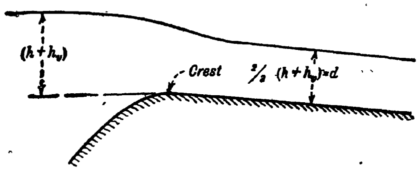


FIG. 81.

Equation (43) is applicable to the discharge through restricted channels such as occur when cofferdams are built part way across a stream, and to temporary openings in dams to pass the stream during construction as indicated in Fig. 82. The depth, d, below the restriction being known, it is desired to determine the elevation of water surface above the restriction. It should be noted, however, that if the depth, d, of water below the restriction is less than $\frac{2}{3}(h + h_v)$,[14] as d' in Fig. 82, the value of d for use in Eq. (43) should be made equal to $\frac{2}{3}(h + h_v)$[14] because the water surface through the restriction will be

[13] Sec. 77.

[14] Or Eq. (44) may be used as it is the equivalent of Eq. (43) for $d = \frac{2}{3}(h + h_v)$.

at that depth (d'' in Fig. 82) as hereinbefore explained. In this case, also, the value of l should be taken from Eq. (42), it being remembered, as explained in this article, that the sum of the two end contractions for narrow restrictions cannot be greater than $2 \times 0.106 = 0.212w$.

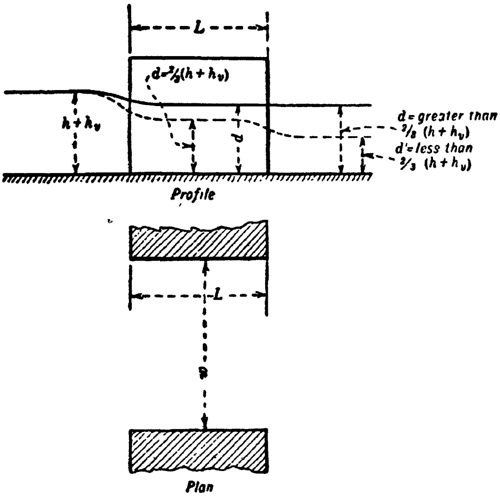


FIG. 82.

75. Measuring Weirs.—From Eq. (38) of Sec. 74, the basic equation for the flow over weirs is:

$$Q = Clh^{3/2}. \qquad (45)$$

The coefficient C varies with the type of crest, the head on the weir, and the depth of the channel of approach. The usual correction for l required on account of end contractions is that given in Eq. (42) of Sec. 74. Most measuring weirs are built without end contractions unless they are made to duplicate exactly a given set of accurate experiments.

A number of equations, based on experimental data, have been proposed for the discharge over sharp-crested rectangular weirs.

Francis' Weir Equation.—The best-known equation is that of Francis,[15] which, for weirs without end contractions, is:

$$Q = 3.31l[(h + h_v)^{3/2} - h_v^{3/2}]. \qquad (46)$$

Eq. (42) is Francis' correction for end contractions. With no velocity of approach, Eq. (46) reduces to the well-known form:

$$Q = 3.31lh^{3/2}. \qquad (47)$$

Francis' equations are subject to considerable error, particularly for high velocities of approach and small values of h. The equation should not be used for accurate weir measurements and is given here only because Eq. (47) is a convenient one to remember for rough approximations.

Fteley and Stearns' Weir Equation.—This equation [16] has the following form:

$$Q = 3.31l(h + \beta h_v)^{3/2} + 0.007l, \qquad (48)$$

where l is obtained from Eq. (42). The value of β is 1.50 for weirs without end contractions and 2.05 for weirs with end contractions.

[15] "Lowel Hydraulic Experiments," by J. P. Francis.

[16] "Description of Some Experiments on the Flow of Water . . ." by Fteley and Stearns. Trans. Am. Soc. C. E., Vol. 12, p. 1, 1883.

Equation (48) is the result of studies of their own experiments and those of Francis. The range of conditions covered by these experiments is:

	Francis	Fteley and Stearns	
		With End Contractions	Without End Contractions
Limit of head	0.6 to 1.6	1.0	0.83 and 1.63 ft.
Height of weir	2.0 and 5.0	3.56	3.17 and 6.55 ft.
Length of weir	8.0 and 10.0	2.30 to 4.00	5.00 and 19.00 ft.
Width of channel of approach	10.0 and 14.0		
Maximum velocity of approach	0.2 to 1.0	0.54	0.6 and 0.8 ft. per sec.

Fteley and Stearns' experiments indicate discharges under low heads about 3 per cent less than those of Bazin.

Bazin's Weir Equation.—Bazin [17] derived his equation from his own experiments. It is written for weirs without end contractions.

$$Q = \left(0.405 + \frac{0.00984}{h}\right)\left(1 + 0.55\frac{h^2}{D^2}\right)lh\sqrt{2gh}. \quad . \quad . \quad (49)$$

The experiments covered the following range of conditions:

Range of head	0.3 to 1.78 ft.
Height of weir	1.16 to 3.72 ft.
Length of weir	1.64 to 6.56 ft.

King's Weir Equation.—King [18] proposes the following equation which, after considerable study, he finds to agree more closely with various experimental data than any of the others:

$$Q = 3.34lh^{1.47}\left(1 + 0.56\frac{h^2}{D^2}\right), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (50)$$

where l is obtained from Eq. (42).

Choice of Type and Method.—As no equations have been derived which will agree exactly with experimental data, precise measurements should be made under conditions identical with those under which some one set of original experiments were made. Bazin's experiments are by far the most complete; but they have not been proved to be more accurate than those of Fteley and Stearns. As the latter indicate discharges for low heads somewhat less than those of Bazin, they have been used extensively for mesaurements where an error, if any, would result in a determined flow less than the actual.

On account of the disagreement between the various proposed equations, the preliminary draft of the "Test Code for Hydraulic Power Plants and Their Equipment," of the American Society of Mechanical Engineers, specifies

[17] Translation by Marichal and Trautwine, Proceedings Engineers' Club of Philadelipha, 1890.
[18] See Bibliography, Sec. 80.

the use of Eq. (45) for sharp-crested rectangular weirs without end contractions. The coefficient C is to be taken from Table XXV which gives values including the correction for the velocity of approach.

TABLE XXV

TEST CODE VALUES OF C FOR VARIOUS HEADS AND HEIGHTS OF CREST

	Height of Crest, P										
Head, h	4	5	6	7	8	9	10	12	14	16	20
1.0	3.376	3.356	3.344	3.335	3.329	3.325	3.322	3.317	3.314	3.311	3.308
1.2	3.391	3.366	3.350	3.339	3.332	3.326	3.322	3.316	3.311	3.308	3.305
1.4	3.409	3.378	3.359	3.346	3.336	3.330	3.324	3.316	3.311	3.307	3.303
1.6	3.429	3.392	3.370	3.354	3.343	3.334	3.328	3.319	3.312	3.308	3.302
1.8	3.450	3.408	3.382	3.363	3.350	3.340	3.333	3.322	3.315	3.309	3.303
2.0	....	3.425	3.394	3.373	3.358	3.347	3.338	3.325	3.317	3.311	3.304

The coefficients in Table XXV are the average of values computed by the equations of Fteley and Stearns, of Bazin, and of Rehbock. The equation of Rehbock is:

$$Q = \left[0.605 + \frac{1}{320h - 3} + 0.08\frac{h}{P}\right]\frac{2}{3}\sqrt{2g}\,lh^{3/2}. \quad . \quad . \quad (51)$$

The following specifications for measuring weirs are given in the Test Code. These specifications have been slightly altered to suit general conditions.

Care shall be taken that smooth flow, free from eddies, surface disturbances or the presence of considerable quantities of air in suspension, exists in the channel of approach. To insure this condition the weir shall not be located too close to the end of the turbine draft tube or other sources of disturbance, and stilling racks shall be used when required.

The channel of approach shall be straight and of uniform cross-section. Where used, stilling racks shall be arranged to give approximately uniform velocity across the channel of approach. The uniformity of velocity shall be verified by current meter or otherwise.

One method of determining the head on the weir, or the elevation of the surface of the flowing water, is by direct observation on it as it flows. The head on the weir may also be observed by hook gages placed in stilling boxes communicating with orifices approximately 1 in. in diameter in the sides of the channel of approach, approximately 1 ft. below the level of the crest, the head being observed independently on both sides of the channel.

The head, h, on the weir shall be measured at a distance of not less than five nor more than ten times the thickness of the stream going over the weir upstream from the weir. The weir shall be sharp-crested, with smooth vertical crest wall, complete crest contraction, and free overfall. A metal crest, free from rust, with sharp right-angle corner on the upstream edge, a crest width of $\frac{1}{8}$ in. and beveled to an angle of 45° on the downstream face, shall be used. The crest shall be carefully leveled.

Complete aeration of the nappe shall be secured and observations of the crest conditions and form of nappe shall be made during the test, to avoid defective conditions such as adhering nappe, disturbed or turbulent flow, or surging. Aeration of the nappe usually requires the construction of air passages leading to the space beneath the nappe.

The sidewalls of the channel shall be smooth and parallel and shall extend downstream beyond the overfall above the level of the crest. Weirs of a

length exceeding approximately twenty times the head (excepting in cases where the velocity of approach is extremely low); or weirs of moderate crest length having high velocities of approach; or those in which the velocity of approach is irregularly distributed, or in which the leading channel is subject to the action of the wind, shall be either subdivided into a number of sections or the head shall be observed not only at both sides, but also at intermediate points across the channel of approach.

The head on the weir shall be measured carefully. A small percentage error in head measurement causes approximately 1.5 times that error in calculated discharge. Considerable errors in the measured head may be made by careless methods of referring the crest level to the head gage scale, and by assuming that a non-level crest is level without knowing the average level. The mean level of the water surface shall be measured, not the crest of small waves or surges existing in all flowing water.

76. Head by Bernoulli's Theorem.—The velocity head, or head required to produce a given velocity, is derived in Sec. 67 and is,

$$\text{Velocity head} = h_v = \frac{v^2}{2g} \qquad (52)$$

According to Bernoulli's Theorem, the general law governing steady flow of water in conduits is as follows:

For steady flow in a conduit, the sum of the velocity head, the pressure head, and the potential head at any point, A, is equal to the sum of the corresponding heads at any upstream point, B, less the frictional resistance between the points A and B.

Expressed mathematically, we have:

$$\frac{v^2}{2g} + h_p + h_e = \frac{v'^2}{2g} + h_p' + h_e' - h_f, \qquad (53)$$

where v and v' = the velocities in feet per second at points A and B, respectively,

h_p and h_p' = the corresponding pressure heads, in feet,

h_e and h_e' = the corresponding potential heads above a common datum plane, in feet, and

h_f = the total frictional resistance or lost head, in feet, between points A and B.

In a closed conduit under pressure, h_p is the height at which water will stand in a piezometer tube, as indicated in Fig. 83, and defines the elevation of the hydraulic gradient above the conduit; and $h_p + \frac{v^2}{2g}$ defines the corresponding elevation of the energy gradient.

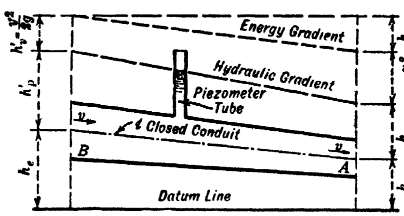


Fig. 83.—Bernoulli's Theorem.

In an open conduit, h_e and h_e' are measured to water surface and h_p and

h_p', measured to the same place, are equal to zero. Therefore the hydraulic gradient for open conduits is at water surface (Fig. 96).

For water-power developments, an additional head, h, must be included in the theorem. This head is that used by the turbines in the generation of power and includes all frictional resistance within the turbine casing, the turbine, and the draft tube, since such losses are chargeable against the efficiency of the turbine. The special case of Bernoulli's Theorem, applicable to water power, is:

$$\frac{v^2}{2g} + h_p + h_e = \frac{v'^2}{2g} + h_p' + h_e' - h_f - h. \quad . \quad . \quad . \quad . \quad (54)$$

For encased turbines, let the points A and B be located at the upper end of the tail race and in the end of the penstock at the turbine casing, respectively.

In the tail race, $h_p = 0$ as it is an open conduit.

The only loss between A and B, not chargeable against turbine efficiency, is that due to the sudden enlargement at the end of the draft tube. If v_d is the velocity at the end of the draft tube, the loss may be assumed to be the difference in the velocity heads in the draft tube and in the tail race, or

$$h_f = \frac{v_d^2}{2g} - \frac{v^2}{2g}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (55)$$

Making the proper substitutions in Eq. (54), we have:

$$h = h_p' + (h_e' - h_e) + \frac{v'^2}{2g} - \frac{v_d^2}{2g}. \quad . \quad . \quad . \quad . \quad (56)$$

Equation (56) agrees with the definition of the Testing Code of the Machinery Builders' Society, given in Sec. 49. for the net head acting on encased reaction turbines.

For reaction turbines in open flumes, h_p' is zero and $\frac{v'^2}{2g}$ is negligible. Therefore, for such installations, the equation is:

$$h = h_e' - h_e - \frac{v_d^2}{2g}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (57)$$

Equation (57) corresponds to the definition of the Machinery Builders' Society, given in Sec. 49, for the net head acting on turbines in open flumes.

For impulse turbines, let the point A be located at the jet where it becomes tangent to the bucket circle, and point B in the penstock at the nozzle casing. After leaving the runner,[19], $h_p = 0$ and $h_v = 0$. The only loss between A and B is that in the nozzle, which is chargeable against turbine efficiency, therefore $h_f = 0$. Consequently, Eq. (54) reduces to:

$$h = h_p' + (h_e' - h_e) + \frac{v'^2}{2g}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (58)$$

[19] Theoretically $h_v = 0$. Actually the water has velocity in a vertical direction necessary to drop to the tail race level, the energy of which is lost, and sometimes it has a horizontal velocity due to imperfections in design, the energy of which is chargeable against turbine efficiency.

This equation corresponds to the definition given in Sec. 49 for the net head acting on impulse turbines.

For the whole development, let A and B be located in the river where the water is returned and where it is diverted, respectively. The pressure head at these points is zero; the velocity at the point of diversion is usually negligible; and that at the point of return is either zero or not available for power. Therefore:

and Eq. (54) reduces to:

$$h = h_e' - h_e - h_f. \quad \quad (59)$$

From the definition of gross head, given in Sec. 49:

$$H = h_e' - h_e,$$

Therefore,

$$h = H - h_f. \quad \quad (60)$$

The net head on the turbine is therefore equal to the gross head, or the dif-

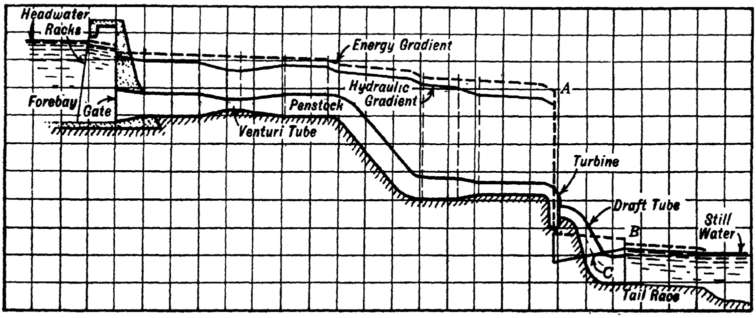


Fig. 84.

ference in elevation between the water surface in the diversion pond and that in the river at the lower end of the tail race, less all frictional losses in the conduits not chargeable against turbine efficiency.

Figure 84 shows the hydraulic and energy gradients for a complete development. The hydraulic gradient is at all places a vertical distance below the energy gradient equal to the velocity head at such places. The energy gradient at any point is a vertical distance below head-water elevation equal to the lost head (total friction losses) between the forebay and that point. Therefore, the hydraulic gradient at any point is a vertical distance below head-water elevation equal to the total friction losses between the forebay and that point plus the velocity head at that point.

The net head available for power is the vertical distance between points A and B since, as hereinbefore explained, the friction losses in the turbine and draft tube are chargeable against turbine efficiency.

It will be noted that the pressure in the draft tube above point C is negative as the hydraulic gradient is below the tube.

The conduit above the turbine should be well below the hydraulic gradient to avoid negative pressures.[20]

77. The Hydraulic Jump and Critical Gradients.—Fig. 85 is a longitudinal section of an open conduit in which the water is flowing at a depth d and at a velocity v. As indicated in Sec. 76, the energy gradient lies at an elevation above the water surface equal to the velocity head, or $h_v = \dfrac{v^2}{2g}$.

From Fig. 85:

$$h = d + \frac{v^2}{2g}.$$

But, if Q is the discharge per foot of width:

$$v^2 = \frac{Q^2}{d^2}.$$

Combining, we have:

$$h = d + \frac{Q^2}{2gd^2},$$

which may be reduced to:

$$Q = 8.02\sqrt{d^2h - d^3}. \qquad (61)$$

Equation 61 indicates that, for given values of Q and h, there are three pos-

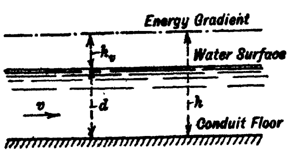


Fig. 85.

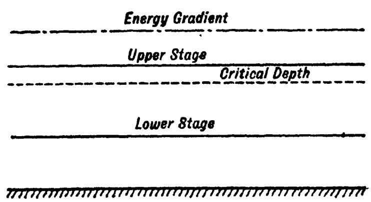


Fig. 86.

sible values of d, one of which, however, is imaginary. Therefore, for a given discharge, Q, and elevation, h, of the energy gradient, there are two depths, d, at which the water will flow, provided the slope of the conduit is suitable for the corresponding velocity and hydraulic radius. This is indicated in Fig. 86. If the energy gradient remains fixed and the discharge increases, the two stages approach each other and are coincident at the critical depth when the conduit is discharging the maximum flow of which it is capable.

For usual conditions at the conduit entrance, the water will flow in the

[20] So far, only steady flow of water has been considered. For unsteady flow due to water hammer, the hydraulic gradient will be much lower than here described and allowance must be made for a lowering of the hydraulic gradient due to water hammer in fixing the elevation of the conduit.

conduit at the high stage as indicated in Fig. 88. Low-stage discharge will obtain for a number of special conditions, two of which are shown in Figs. 89 and 91.

As a practical example, let $h = 10.0$ ft. The curve of Fig. 87 has been plotted by substituting this value of h, together with various values of d, in Eq. (61), the width of conduit used being unity. This curve clearly shows the alternate depths for each discharge of which the conduit is capable with a constant value of $h = 10.0$ ft. It also shows that the maximum possible discharge is 97.7 sec.-ft. corresponding to a depth, d, of $\frac{2h}{3}$ or 6.666 ft. where the two stages are coincident.

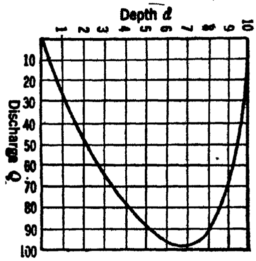


FIG. 87.

That the maximum discharge occurs for $d = \frac{2h}{3}$ may be proved by making the first differential of Q with respect to d in Eq. (61) equal to zero, whence:

$$\frac{(dQ)}{(dd)} = \frac{\overline{8.02}^2}{2Q}(2dh - 3d^2) = 0.$$

From which:

$$d = \frac{2h}{3} = 2h_v. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (62)$$

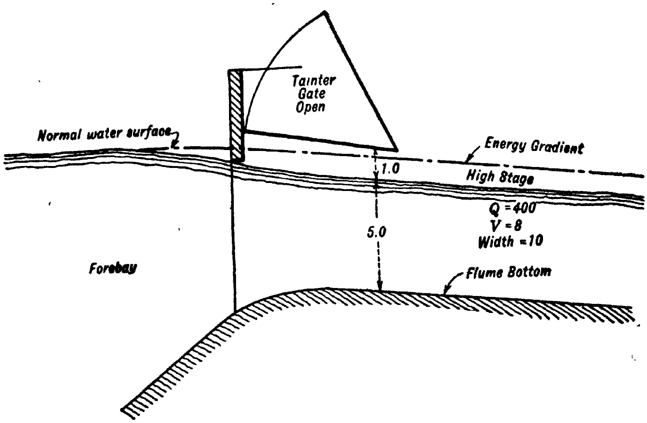


FIG. 88.

The depth $d = \frac{2h}{3}$ is called the critical depth. Depths near the critical depth should be avoided in open-conduit design for, under such conditions, very slight irregularities in the conduit section may cause the water to fluc-

tuate between the two stages with great disturbance and increased friction.

A variety of circumstances will cause the water surface to change from a high to a low stage or *vice versa*. For a drop from the high to the low stage, the most common examples are for flow over spillways (Fig. 91) and for high-velocity discharge through orifices (Fig. 89). Water flowing in an open conduit at the low stage will continue to flow at that stage if the slope is sufficient to overcome the friction corresponding to low-stage velocity. Additional friction, however, eventually will cause the water to rise to the upper stage, and such a rise is known as the hydraulic jump, or standing wave.

Figure 88 represents a flume 10 ft. wide, designed to carry 400 sec.-ft. with normal water surface in the forebay and the entrance gate wide open. It requires a depth of 5 ft. and a velocity of 8 ft. per second. The water flows at the high stage.

Figure 89 represents the conditions of flow during high forebay level when

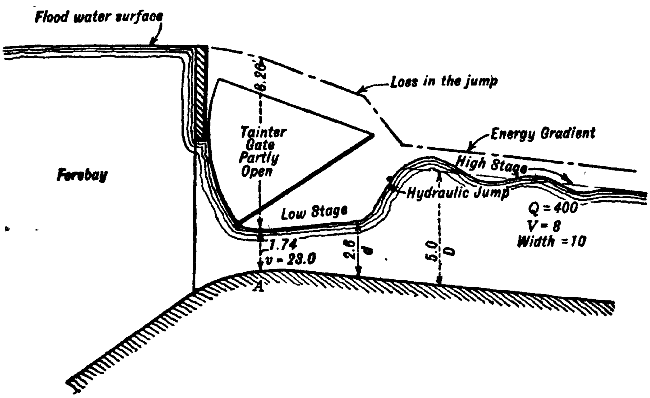


FIG. 89.

the gate is only partly open to limit the flow to the required 400 sec.-ft. The water issues from the gate at the low stage and, if the flume had sufficient slope to overcome friction at this high velocity, the water would continue to flow at this depth. However, the flume has a slope sufficient only for the low velocity and large hydraulic radius of the high-stage flow, and therefore the friction loss is greater than the slope of the conduit, and the depth of water increases. As, for a given high-stage depth and velocity, the jump can happen only for one depth of low stage, the jump occurs when the low-stage depth increases to this value. The equation for the hydraulic jump, as given by A. H. Gibson [21] and others is:

$$D = \sqrt{\frac{2v^2d}{g} + \frac{d^2}{4}} - \frac{d}{2}, \quad \ldots \ldots \ldots \quad (63)$$

[21] "Hydraulics and its Applications," Art. 86, by A. H. Gibson. John Wiley and Sons, 1908.

where d = the depth of the low stage in feet (Fig. 89);
D = the depth of the high stage in feet;
g = the acceleration of gravity = 32.2, and
v = the velocity at the low stage in feet per second.

Usually, however, the known quantities are D, the depth at the high stage, and V, the velocity at the high stage. Frequently, therefore, a more convenient form for the hydraulic-jump equation is:

$$d\left(\frac{2V^2D}{g} + D^2\right) - d^3 = \frac{2V^2D^2}{g}. \qquad (64)$$

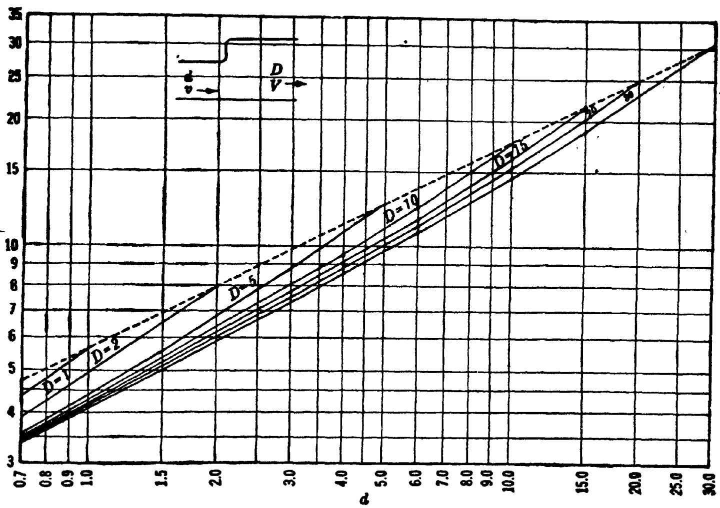


FIG. 90.—Characteristics of the Hydraulic Jump.

Experiments [22] on the hydraulic jump agree very closely with the foregoing equations.

The hydraulic jump can occur only if the flow is at the low stage, i.e., if the water surface is below the critical stage. From Eq. (62), the flow is at the low stage if

$$d = < 2h_v.$$

That is:

$$d = < 2\left(\frac{v^2}{2g}\right).$$

Transposing we find that the hydraulic jump is possible only if

$$v^2 = > gd. \qquad (65)$$

Equation (64) is indicated diagrammatically in Fig. 90. In most practical

[22] See "The Hydraulic Jump as a Means of Dissipating Energy," by Riegel and Beebe. Technical Reports, Part III, Miami Conservancy District, Dayton, Ohio, 1917.

cases the depth and velocity at the high stage is known and it is desired to know the low-stage depth at which the jump will occur.

Using the conditions of Fig. 89 we see from Fig. 90 that, for $D = 5$ ft. and $V = 8$ ft. per second, the depth, d, necessary for the jump is 2.6 ft. The dotted line of Fig. 90 indicates the limiting conditions of the hydraulic jump from Eq. (65).

The distance between the gate and the hydraulic jump can be determined by the principle of varied flow described in Sec. 79.

Figure 91 shows the hydraulic jump at the toe of a spillway dam. The water passes from the high to the low stage at the crest and flows down the face of the dam and into the lower river at the low stage. For a given discharge, the depth, D, of the water below the dam and its velocity, V, must be known, and the depth, d, of water at low stage which will cause the hydraulic jump can be obtained from Eq. (64) or Fig. (90). If the depth, d, at the toe

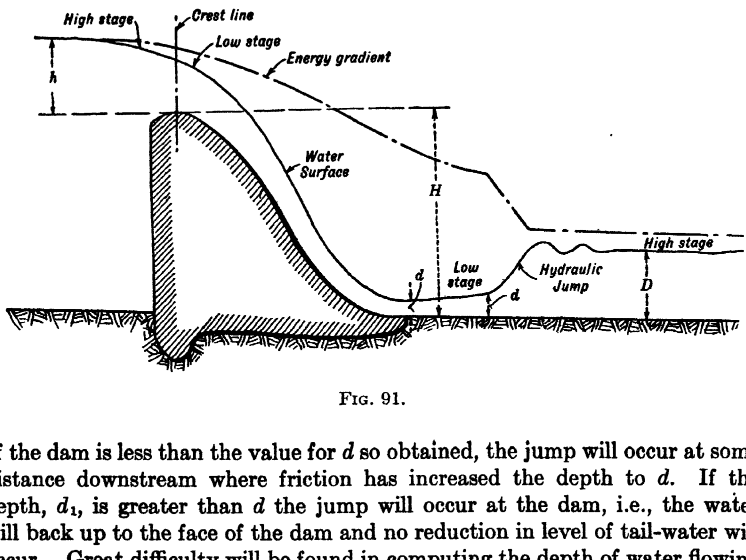


FIG. 91.

of the dam is less than the value for d so obtained, the jump will occur at some distance downstream where friction has increased the depth to d. If the depth, d_1, is greater than d the jump will occur at the dam, i.e., the water will back up to the face of the dam and no reduction in level of tail-water will occur. Great difficulty will be found in computing the depth of water flowing down the face of the dam if the ratio of height of dam to head on the crest is great. This is because of the lack of knowledge regarding friction of water at high velocities. The thickness of the flowing water at and near the crest can be obtained from Fig. 126 and Table XXXII; but the thickness near the toe can only be approximated. It will be somewhat greater than that indicated by Fig. 126 because that figure shows the thickness without friction.

For excellent treatises on the hydraulic jump see " The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures," by Julian Hinds, Eng.-News-Record, Vol. 85, p. 1034, and " The Hydraulic Jump as a Means of Dissipating Energy," by Riegel and Beebe, Technical Reports, Part III, Miami Conservancy District, Dayton, Ohio, 1918.

78. The Hydraulic Bore and the Suction Wave in Open Conduits.—Fig. (92) is a profile of a typical open conduit. The normal depth at constant flow, q, is d and the corresponding velocity is v. If the flow out of the lower end is suddenly stopped by a closure of the turbine gates, the water surface at the lower end will immediately rise and the hydraulic bore, or pressure wave, AB, will advance up the conduit with a velocity of propagation, a. The discharge, q, rejected by the turbines will fill the space $ABCE$ and create automatically an excess head or force to decelerate the flow in the conduit progressively as the bore advances, the velocity below the bore being zero and that above the bore being v.

A knowledge of the height, AB, of the bore is necessary in order to fix the height or superelevation of the sides of the conduit.

The hydraulic bore is simply another form of the standing wave, described in Sec. 77. In order to apply Eq. (63) for the standing wave to the case of the hydraulic bore, it is necessary to remember that the velocity of flow, v, is the velocity past the standing wave, and that in the case of the hydraulic bore

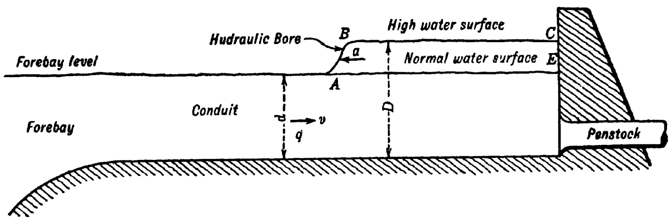


FIG. 92.—Hydraulic Bore, without Friction.

advancing with a velocity a, the velocity of flow past the wave is $v + a$. Therefore, an equation for the hydraulic bore can be obtained by substituting $v + a$ for v in Eq. (63).

But,

$$a = \frac{q}{D - d} = \frac{dv}{D - d}.$$

Therefore,

$$v + a = \frac{vD}{D - d}.$$

And, making this substitution for v in Eq. (42), we have:

$$D = \sqrt{\frac{2d}{g}\left(\frac{vD}{D - d}\right)^2 + \frac{d^2}{4}} - \frac{d}{2}, \quad \quad (66)$$

which is the equation of the hydraulic bore.

This equation can be solved only by trial. The value of d is entered in the equation, and then different values of D are adopted and substituted until the second term of the equation equals the value of D adopted. To assist in the solution of this equation, an approximate value of D may be obtained from

the following empirical equation by Kennison,[23] which is in more convenient form than Eq. (66):

$$D = \frac{v\sqrt{d}}{5} - 0.99d. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (67)$$

So far, friction in the conduit has not been considered. Figure (93) shows a practical case where there is a slope in the normal water surface. In such cases the discharge rejected by the turbines must also fill the volume *BCF* which grows larger as the bore advances toward the forebay. This causes a reduction in the velocity, *a*, and hence a reduction in depth, *D*, because the residual velocity between the bore and the lower end of the conduit is not zero but has a value equal to that required to fill the ever-increasing volume in the triangle *BCF*. However, this residual velocity must be eliminated after the bore reaches the forebay and thus an additional rise of water surface will be required. Thus we have for a practical case a tendency for both a reduction and an increase in the value *D*, both of which are relatively small and may be considered to counteract each other.

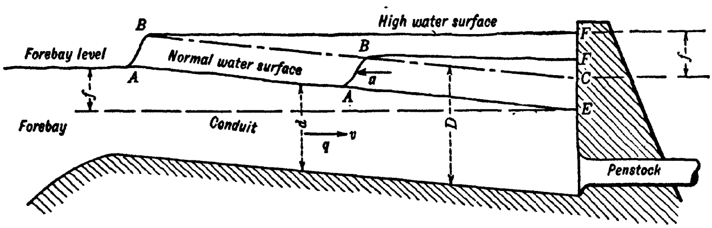


FIG. 93.—Hydraulic Bore, with Friction.

Therefore, for the practical case, the maximum depth of water in the conduit may be considered to be equal to *D* from Eq. (67) at the forebay and, at the lower end of the conduit, equal to *D* plus the original conduit friction loss, *f* (Fig. 93). The high-water surface would then be at the same level throughout the conduit.

A sudden increase in the demand for water in the lower end of an open conduit will result in a sudden drop in the water level at the lower end. The lower level and the resulting increase in slope serve to accelerate the water to meet the increased demand. This phenomenon is known as the suction wave and is the direct opposite of the hydraulic bore, except that it does not travel up the conduit as a constant wave; instead, the slope of the wave becomes flatter as it moves towards the upper end. Practical interest, in this case, is at the lower end of the conduit where the minimum depth, *d*, must be known, for expected load changes. R. D. Johnson[24] gives as the equation of the suction wave:

$$D - d = (v_2 - v_1)\sqrt{\frac{d}{g}} + \frac{(v_2 - v_1)^2}{4g}.$$

[23] Karl R. Kennison, Trans. Am. Soc. C. E., Vol. LXXXI, p. 119 (1917).
[24] Trans. Am. Soc. C. E., Vol. LXXXI, p. 113, (1917).

But,

$$v_2 = \frac{q_2}{d}.$$

Therefore,

$$D - d = \left(\frac{q_2}{d} - v_1\right)\sqrt{\frac{d}{g}} + \frac{\left(\frac{q_2}{d} - v_1\right)^2}{4g}. \quad . \quad . \quad . \quad . \quad . \quad (68)$$

79. Varied Flow.—The flow in rivers and conduits having a variable cross-section or slope is known as varied flow. The back-water curve, upstream from a dam, as indicated in Fig. 95, is a typical example of varied flow.

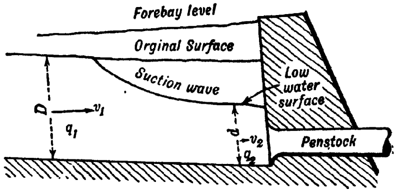


FIG. 94.—Suction Wave.

Varied-flow problems can be approached most readily by the use of Bernoulli's Theorem, described in Sec. 76; and Eq. (53), for open-water conditions, can be written:

$$h_v + h_e = h_v' + h_e' - h_f,$$

whence,

$$h_e' - h_e = h_v - h_v' + h_f. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (69)$$

Where the letters are as indicated in Fig. 96.

The river (or conduit), the water surface of which is to be determined, is divided into a number of reaches as shown in Fig. 95, although the reaches are usually much more numerous than there indicated. Each reach should include a length that is as nearly uniform as possible in section and slope.

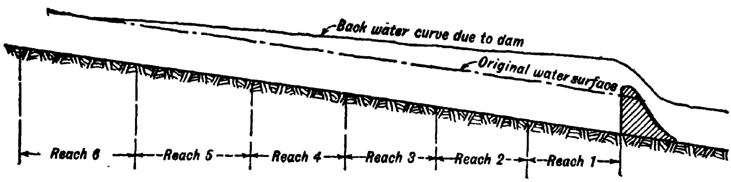


FIG. 95.—Back Water Curve.

In other words, each reach should terminate at a point where there is a decided change in section or slope. Each reach is then investigated separately. The more numerous the reaches, the more accurate will be the determination of the slope.

Let Fig. 96 indicate a typical reach. The first reach to be investigated is that adjacent to the controlling section, which is the section next to the dam for back-water curves. In Eq. (69) h_e and $\frac{v^2}{2g}$ at the dam can be determined

for any given flow. A value for h_e' is first assumed and $h_v' = \frac{v'^2}{2g}$ calculated. The equation is then solved for a tentative value of h_f which remains to be checked.

With the value of h_e' first assumed, values of the hydraulic radius and velocity at each end of the reach are calculated and averaged. With these average values, and the coefficient of friction applicable to the channel,[25] the slope and hence the total friction loss [26] for the length, l, of the reach are calculated.

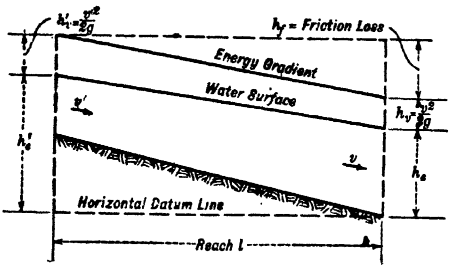


FIG. 96.

If this value of h_f does not agree with that tentatively determined from Eq. (69), new assumptions of h_e' must be made and the calculations repeated until an agreement is reached.

Values so determined for the upper end of the first reach are to be used for the lower end of the second reach.

The controlling section of the flow preceding the hydraulic jump, as indicated in Fig. 89, is at point A, where the depth and velocity are known. The first reach to be investigated, therefore, is that adjacent to point A, and the reaches are investigated successively downstream instead of upstream as in the case of back-water curves, previously described. It will be noted that, for such cases, the friction loss between point A and the jump is greater than the slope of the bottom of the conduit, so that frequently the water surface just before the jump is higher than at point A.

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3. Calculation of Flow in Open Channels, by I. E. Houk. Technical Reports, Part IV, Miami Conservancy District, Dayton, Ohio, 1918.
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8. The Flow of Water in Dredged Drainage Ditches, by C. E. Ramser. Bulletin No. 832, U. S. Dept. Agr., 1920.

[25] Table XXII.

[26] This should include the loss for obstructions and as bridge piers, etc., and for sudden enlargements and contractions. See Sec. 72.

9. Some Better Kutter's Formula Coefficients, by R. E. Horton. Eng. News, Vol. 75, pp. 373 and 863, 1916.
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11. Gagings in the Concrete Conduit of the Umatilla Project, by E. G. Hopson. Eng. Record, Vol. 64, p. 480, 1911.
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CHAPTER X

GENERAL DESIGN

By William P. Creager

81. General.—The usual ultimate object in the design of a hydro-electric power development is the production of power which, within a specified period, will give the greatest return on the investment.

The greatest return on the investment is not necessarily limited to the net profit from the sale of power from the development in question; to this should also be added the increased return from other developments that may be benefited by its installation. A development designed to assist in carrying the peak load of the system, due to insufficient installation or pondage at other plants, might never show a profit if considered alone; but its addition to the system would enable the other plants to produce more actual energy than formerly, and hence the resulting increased revenue for this power should be credited to the return on the investment for the peak-load plant.

The fulfilment of the aforestated ultimate object requires careful consideration of the following influencing factors, coordinated as indicated in Chapter XXXIII.

(*a*) Cost of the development and interest rates on borrowed money;
(*b*) Cost of operation, maintenance, and repairs;
(*c*) Cost of taxes, insurance, management, and other overhead expenses;
(*d*) Rate of depreciation;
(*e*) The amount and duration of the various classes of power produced;
(*f*) Reliability and class of service as affecting the sale value of the power;
(*g*) The length of the non-producing period of construction.

All of these factors are inter-related. A change in design resulting in the adoption of a better class of structures and apparatus would increase the cost of the development and therefore the taxes, insurance, and interest charges; but it might prove desirable if it resulted in a sufficient reduction in annual cost of operation, maintenance, repairs, and depreciation, or a sufficient increase in the reliability and amount of power produced. Consequently, the engineer is interested not only in the first cost but also in the execution of a design which, in conjunction with all other factors, will be best suited to the conditions governing the ultimate object of the development. Most of the recent major improvements in the design and fabrication of apparatus and auxiliaries are those which make for continuity of service; and in many instances the improvements to both structures and apparatus which have

met with the greatest favor have resulted in increased cost of development.

A hydro-electric development may be divided into the following major divisions:

(a) The diversion dam;
(b) Conduits to convey the water to the turbines;
(c) The turbines and governors;
(d) Conduits to carry the water away from the turbines;
(e) Electrical apparatus to convert the mechanical energy of the turbines to electrical energy at the voltage required for transmission;
(f) Housing and supports for the hydraulic and electric apparatus;
(g) Transmission system to convey the electrical energy to its point of ultimate use, including frequently terminal transformers to lower the voltage for local distribution.

These divisions are, of course, common to all hydro-electric developments; but the almost unlimited possibilities of their variation in detail and cost for each project give rise to the need for great experience and the best judgment. It is the function of the engineer to effect an arrangement which will meet most nearly the conditions hereinbefore given, since these conditions govern the ultimate object of the development.

Chapter XXXIII describes in detail the usual method of estimating the value of a projected development. The estimate is usually made in the form of a prospectus, as indicated in the tables of that chapter, to show the anticipated return on the investment. A variation in the type of apparatus or in the details and arrangement of the divisions of the development frequently alters all the items of the prospectus and the estimated return. It is, therefore, necessary to investigate by similar means a number of possible general schemes, as well as different types and sizes of conduits and other structures and apparatus, before the final arrangement of development is adopted.[1]

82. Choice of Site and Type of Development.—Standard hydro-electric practice, as indicated by data derived from published records of existing developments, is inadequate as a guide to the inexperienced designer. Few such records include the line of reasoning which led to the adoption of the various assumptions and the types of apparatus and structures used in the design.

Careful consideration of local conditions, the value of power, the cost of the development, the nature of the market, and many other factors affecting economy, adaptability, and successful operation, is a necessary preliminary to the making of many of the assumptions upon which the design is to be based.

When more than one site is available, the most favorable site, other things being equal, is the one providing:

(a) The minimum cost of development per unit of power;
(b) The minimum unproductive period of construction;

[1] See Sec. 87.

(c) The amount of power most nearly equal to that desired;
(d) The shortest transmission line with its consequent lowest cost and least liability to interruption of service;
(e) The most reliable records of stream flow;
(f) The greatest possible future increase in output due to possible future storage;
(g) The most strategic position for possible future growth of market;
(h) The greatest opportunity for additional developments close by, tending to centralize operation and effect a reduction in the required number of spare parts, machine shop equipment, etc.
(i) The most accessible site for transportation of repair and maintenance equipment during operation.

The choice of type of development is frequently the greatest problem confronting the engineer. While some projects permit of only one general

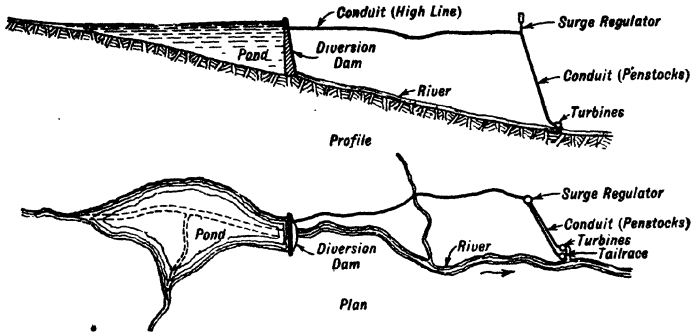


Fig. 97.

type of development, others may possess features which allow practically unlimited combinations of types of structures. In such cases the choice can be made only by careful estimates of cost and studies of other governing features of each type.

Figure 97 shows diagrammatically a basic type of development, the component parts of which are common to all developments, without exception. By this is meant that every development must possess a diversion structure, a conduit, a turbine, and a tail race, although the forms of these may be quite variable.

Where water is plentiful, the diversion structure may be a short wing dam, which projects only a short distance from the banks to deflect the water into the conduit. Usually, however, the diversion dam extends from bank to bank, and it is frequently supplemented by other dams or dikes to prevent the pond from overflowing into another watershed.

As indicated in Fig. 97, the dam may be located at any favorable place on the river between the turbine and the upper end of the pond, simply by varying its height and the length of the conduit.

Other things being equal, the dam should be located as close to the turbine as possible, thus providing the maximum size of pond and the shortest length of conduit. However, as the cost of the dam increases approximately with the third power of its height,[2] a shifting of the dam toward the turbine will usually increase its cost, unless a particularly favorable dam site will compensate for the increased height.

To compensate for this increased cost of the dam, there would be a reduction in the installation and maintenance cost of the conduit, a reduction in the head lost in friction, a reduction in the cost of surge regulation, and a desirable increase in the size of the pond.

For low-head developments, the dam is usually located at the lowest point in the stretch of the river to be developed, and the penstock and turbine are located within the dam or adjacent to it. The power house would then be similar to the types shown in Figs. 100 and 101.

The conduit system may be divided into two parts, namely, the "high-line conduit" and the "penstock." The high-line conduit is that portion which, in general, follows a grade at or as close as possible to the elevation of the low-water hydraulic gradient, as in Fig. 97. The penstock connects the lower end of the high-line conduit with the turbine, as shown.

The high-line conduit may consist of a canal as in Fig. 102, a flume as in Fig. 103, one or more pipe lines as in Fig. 104, a tunnel as in Fig. 105, or a combination of these as in Fig. 106.

For the penstock, a tunnel, as in Fig. 107, or more frequently one or more pipes, as in Fig. 104, are provided.

If the high-line conduit is a pipe, its usual designation is "pipe line" as distinguished from the "penstock" if the latter is also a pipe.

In very low-head developments, the penstock is eliminated and the high-line conduit is an open flume in the end of which the turbine is set as in Fig. 108.

Frequently a high-line conduit is merged with or supplemented by an open body of water at a ravine or valley crossing, where it is cheaper to build a dam than to provide a conduit over or around the depression. Such an open body of water is in reality a part of the conduit system and, if it has sufficient capacity to provide for weekly fluctuations in demand, the conduit above it need be designed to carry only the average weekly flow instead of the peak flow. It therefore has an additional economic advantage in the reduction of the cost of the conduit above it. An open body of water at the lower end of the high-line conduit is called the "terminal reservoir" or, more properly, the "terminal pond."

If the high-line conduit is long, it is usually provided at its lower end with a surge tank, terminal pond, or other surge-regulating device, or provision is made for spilling water when the turbines are suddenly shut down.

The surge-regulating device on the Brown's Falls development is a surge tank, as indicated in Fig. 104. That at the Queenstown-Chippewa development is a terminal pond, as shown in Fig. 102. The flume of the Ocoee

[2] The volume increases exactly as the third power of the height for a triangular section of dam and a triangular cross-section of the river valley at the site.

No. 2 development, Fig. 103, is provided near its end with a terminal pond formed by a dam containing a syphon spillway.

In the case of a long stretch of river to be developed for power, the question frequently arises as to whether the development is to be made in a single stage, as in Fig. 97, or with two or more individual plants, as in Fig. 98. There are only two reasons for adopting more than a single plant and these are as follows:

(a) A single plant may be too large for the existing market. In this case the whole head may be developed in a series of plants, constructed one by one to keep pace with the growth of the market.

(b) Two or more plants may show a better return on the investment than a single one.

With unlimited market, two or more developments ordinarily would be made only if the construction cost were considerably less than that of a single development. With nearly equal cost, the single development is much to

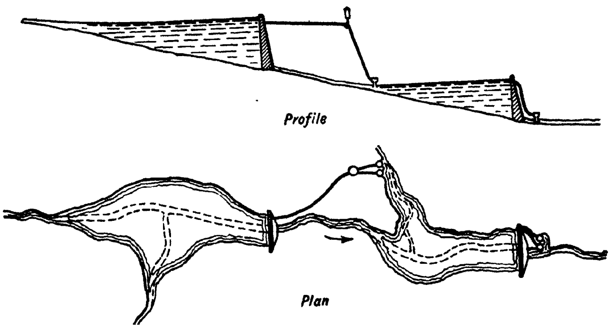


FIG. 98.

be preferred because of its requiring only one corps of operators and less duplication of apparatus.

For two adjacent developments, as in Fig. 98, it is advisable to put the spillway crest of the lower dam higher than the water surface in the tail race of the upper development, if the lower pond is to be drawn down. This avoids loss of head between the two developments when the lower pond is drawn down.

The water is sometimes diverted from one watershed to that of an entirely different river; but more frequently a diversion of this kind is made to the watershed of a tributary stream. Fig. 109 shows the layout of the Soft Maple development, in which the water from the diversion pond on the main river is carried to a terminal pond on an adjoining watershed of a tributary stream, and then rediverted from the terminal pond to a canal and penstocks leading to a power house on the main river. An alternative arrangement would have been to carry a pipe line directly from the diversion dam to the upper end of the penstocks; but the arrangement adopted was found to be more economical as to cost, and to provide more desirable pondage and less depreciable structures.

Figure 99 shows several types common to low-head developments, of which there are, of course, a number of variations and combinations. Sketch *A* shows the commonest form, consisting of a combined dam and power house.

If there is a fall in the river below the site adopted for the dam, a tail race may be excavated, as indicated in Sketch *B*, protected by a river wall extending all or part of its length to prevent overflow into it during periods of high water. Or this fall may be utilized, as indicated in Sketch *C*, by placing the

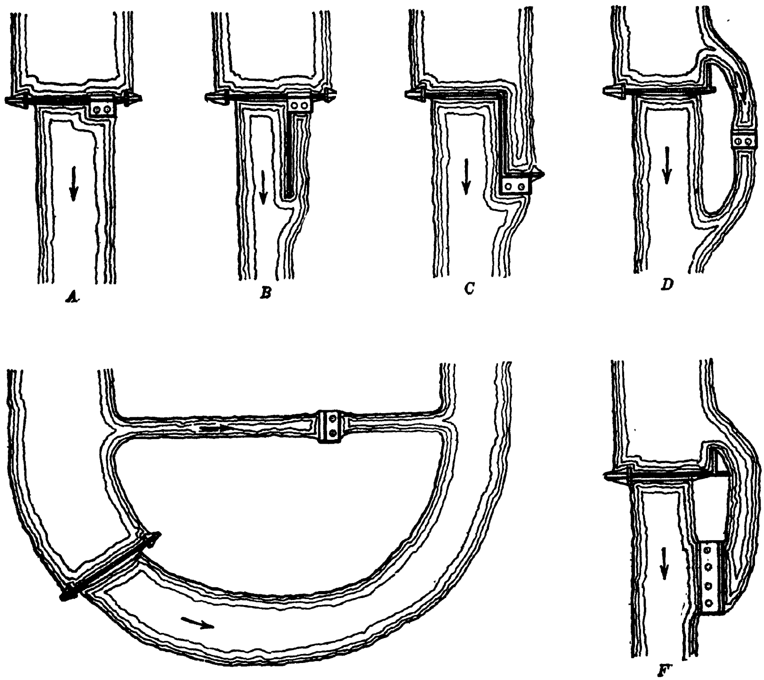


Fig. 99.

power house at the foot of the fall and connecting it to the dam by means of a canal formed by a wing or extension of the dam or some type of flume.

Frequently an arrangement like that shown in Sketch *D* is advisable, if the topography is favorable, particularly if an increase in the spillway length of the dam is desirable.

The arrangement shown in Sketch *E* is quite common at places where there is a sharp bend in the river. In such cases the conduit between the pond and the power house is often a tunnel if the land is high.

When a large number of units is employed, it is sometimes necessary to adopt the scheme indicated in Sketch *F*; but this has the objectionable feature of flow in the canal across the racks instead of directly toward them,

resulting in several times the loss of head for the same average velocity of flow through the racks.

If two or more types of development show relatively little difference in

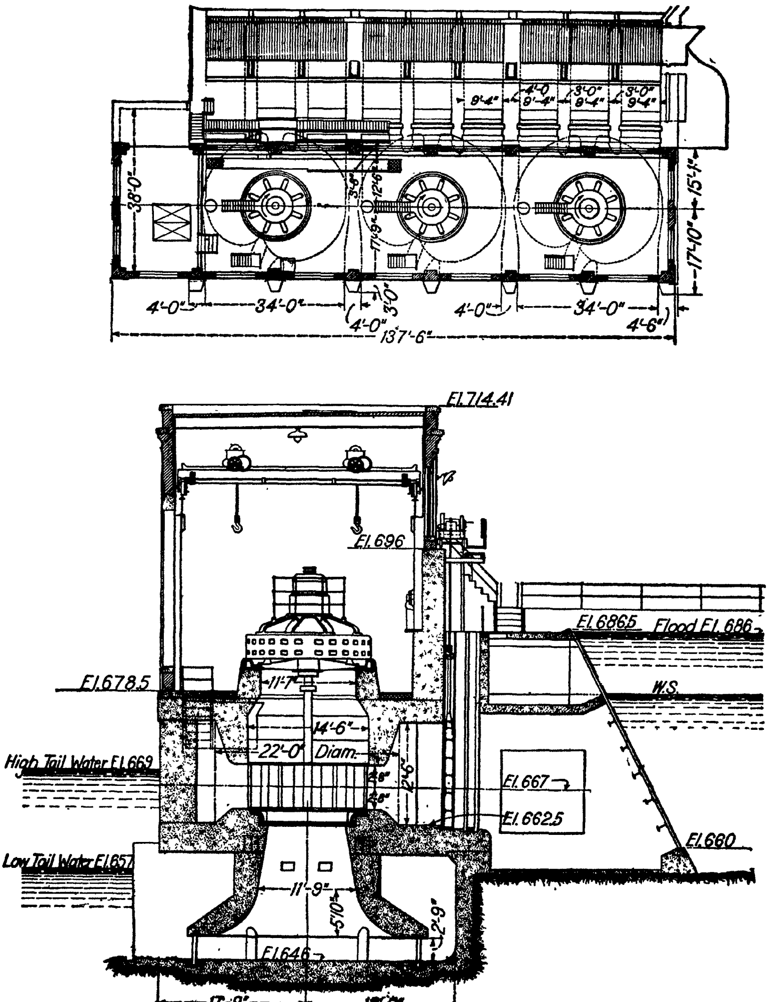


Fig. 100.—Herrings Development. Black River, near Watertown, N. Y. Northern New York Utilities, Inc. 3 Units—2250 H.P. Each at 19.5 Ft.-Head.

economic advantage when considered in connection with the ultimate object of the development, it is always advisable to adopt that type which requires the shortest conduit system. Continuity of service is the most essential fea-

ture in modern operating requirements, and considerable sums have been spent in recent developments to eliminate, in so far as possible, those features which are most subject to deterioration, and most likely to require excessive maintenance and repairs and constant attention.

Long canals and flumes are very undesirable in cold climates on account of ice conditions. The serious troubles frequently caused by ice are described in Sec. 176. In all climates, canals frequently require constant attention on account of silting, sloughing banks, growth of weeds, burrowing animals, and seepage losses.

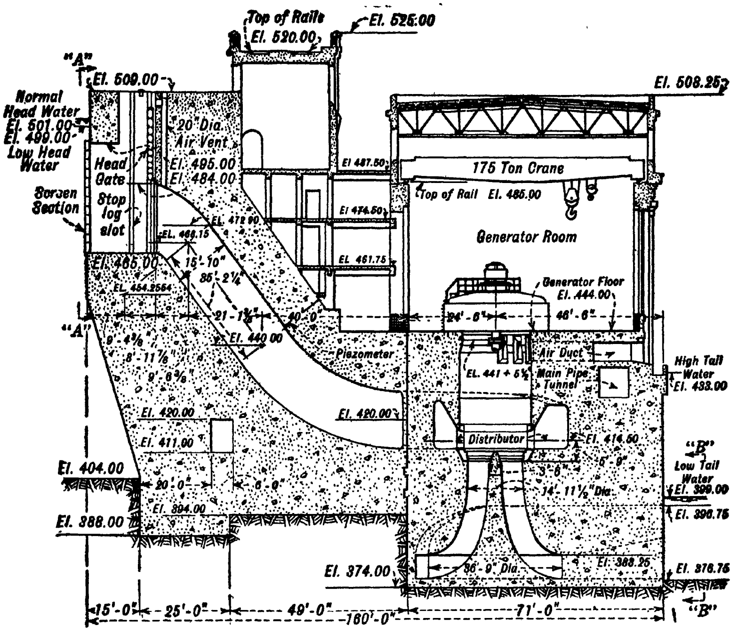


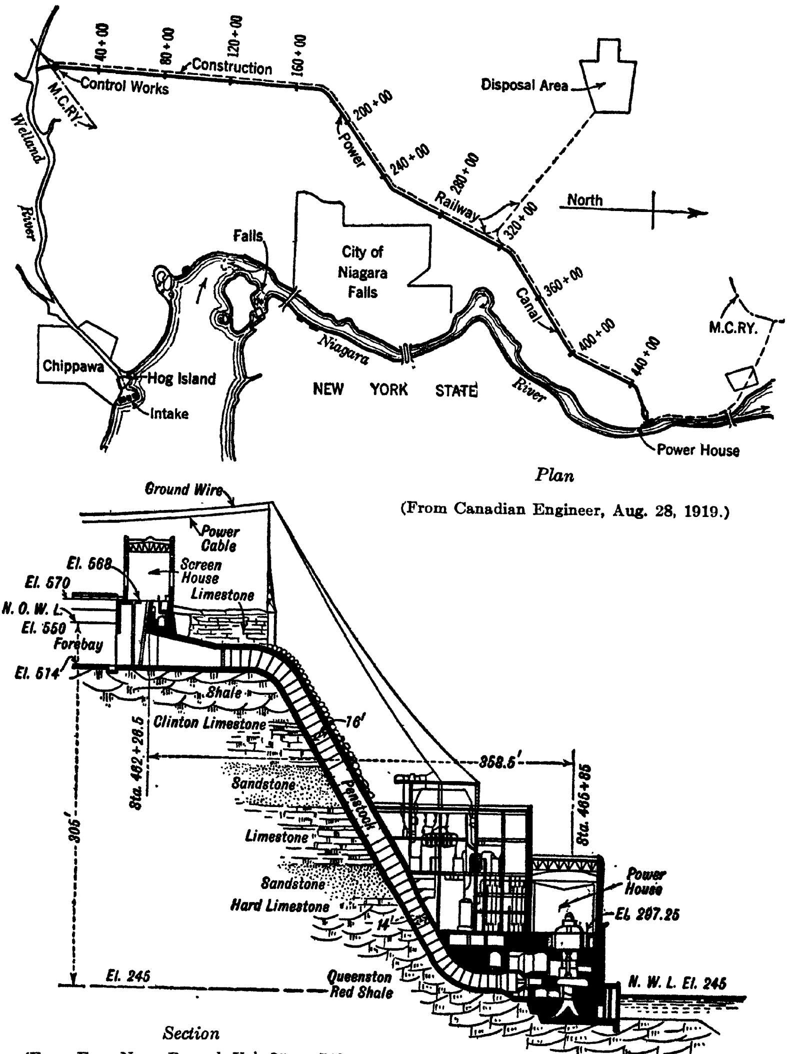
FIG. 101.—Wilson Dam, Tennessee River Muscle Shoals, Ala. Section through One of the Four 30,000 H.P. Units. Eventual Capacity of Plant, 500,000 H.P.

Long pipes, while not as objectionable as canals, are frequently a source of trouble. Several have collapsed during recent years because of faulty design. Leaky or defective expansion joints are often a source of worry and expense, and exposed steel pipes in cold climates accumulate ice sheets which, when warm weather sets in become loose and are likely to block the turbine. Tunnels give the least trouble but are usually the most expensive type of conduit.

The penstocks should be as short as possible, to avoid excessive water hammer and to obtain the best turbine regulation during load changes.

It is obvious that a solid concrete dam of which the power house is an integral part, as in Figs. 100 and 101, eliminates, to the greatest possible extent, the undesirable features of hydro-electric developments. Any devia-

tion from this type is objectionable from an operating point of view and should not be undertaken unless great economy is thereby effected.



Plan

(From Canadian Engineer, Aug. 28, 1919.)

Section

(From Eng. News Record, Vol. 85, p. 746.)

FIG. 102.—High Line Conduit is a Canal. Queenston-Chippewa Development, Niagara River, Ontario. Five 50,000 H.P. Units. Eventual Development 500,000 H.P.

As more extensively discussed in Sec. 39, adequate pondage [1] is usually an essential requirement of an economical development, being necessary in

[1] See definitions of "pondage" and "storage" in Sec. 30.

order that the flow of the stream may be regulated to suit the load demand. Many an otherwise attractive site has been abandoned because of lack of adequate pondage.

The probability of deposits of silt destroying the effectiveness of both pondage and storage reservoirs should be carefully investigated, as excessive silting has frequently proved serious, and no effective means for removing enough of the silt to restore the usefulness of the reservoirs has yet been

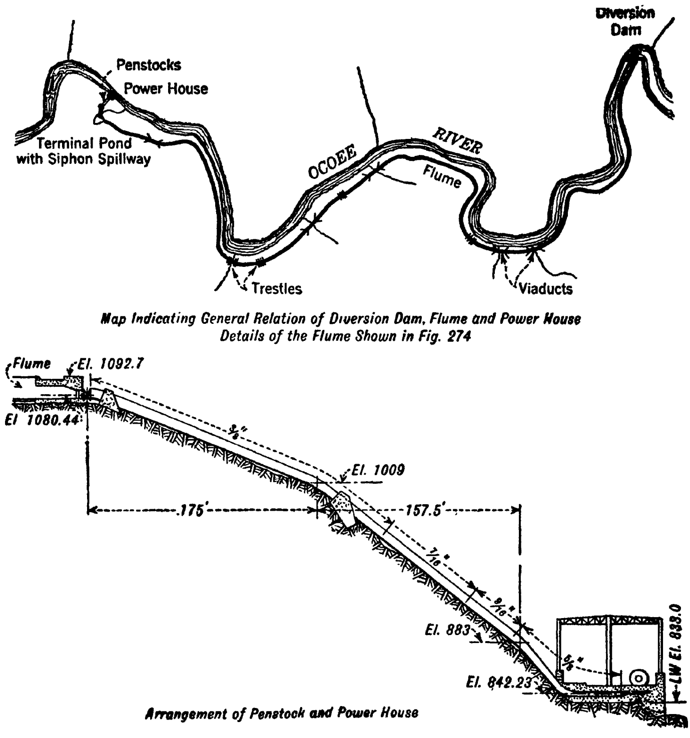


FIG. 103.—Ocoee No. 2 Development, Tennessee River above Parksville, Tennessee. Tennessee Power Co. 3–10,000 H.P. Units at 250 Ft.-Head.

found. If a number of storage reservoir sites are available, sufficient funds should be set aside from the earnings of the plant to build new reservoirs to keep pace with the reduction of available storage capacity due to silting.

Much study can profitably be given to the choice of type and site where a number of sites for development are available, and particularly where storage reservoirs are involved. A deficiency of primary power, resulting from insufficient natural flow during low-water season, necessitates the construction of storage reservoirs to increase the flow, or of steam or other power plant

auxiliaries to supplement deficient power, as more fully explained in Sections 45, 51 and 56. In those sections was mentioned also the feasibility of high-

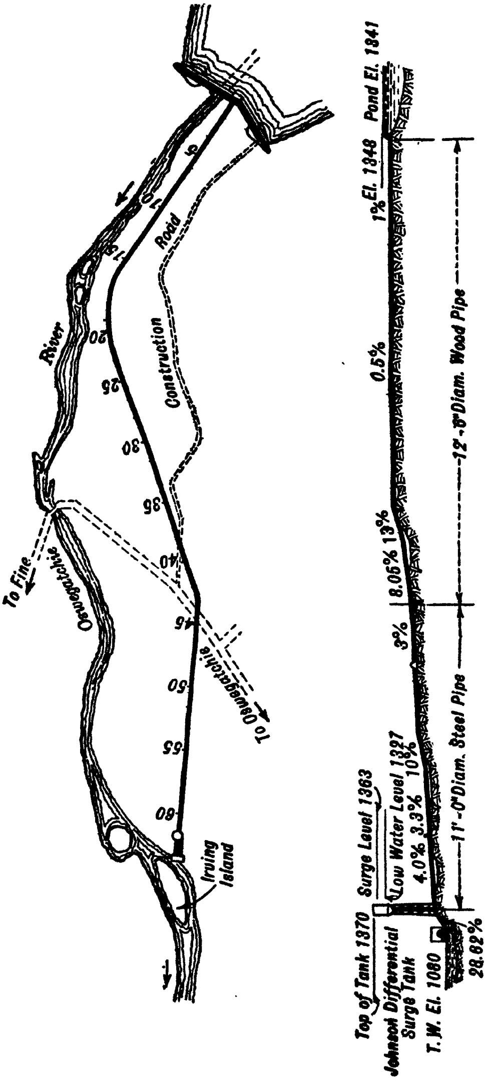


FIG. 104.—Browns Falls Development, near Oswegatchie, N. Y. Northern New York Utilities, Inc. 2 Units each 11,500 H.P. at 261-Ft Head.

head plants with ample storage, operating only during the low-water season, to supplement the output of low-head unregulated streams.

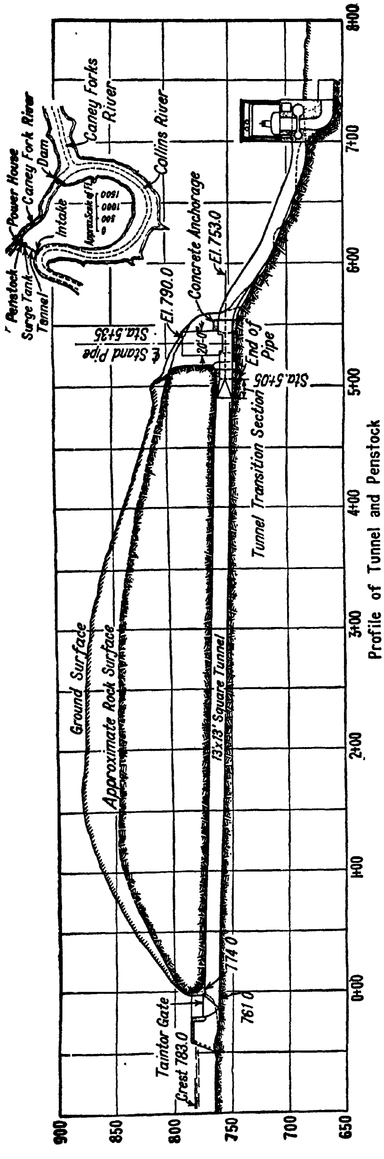


FIG. 105.—Great Falls Development, near Great Falls, Tenn. Tennessee Power Co. One 12,900 H.P. Unit at 110 Ft.-Head.

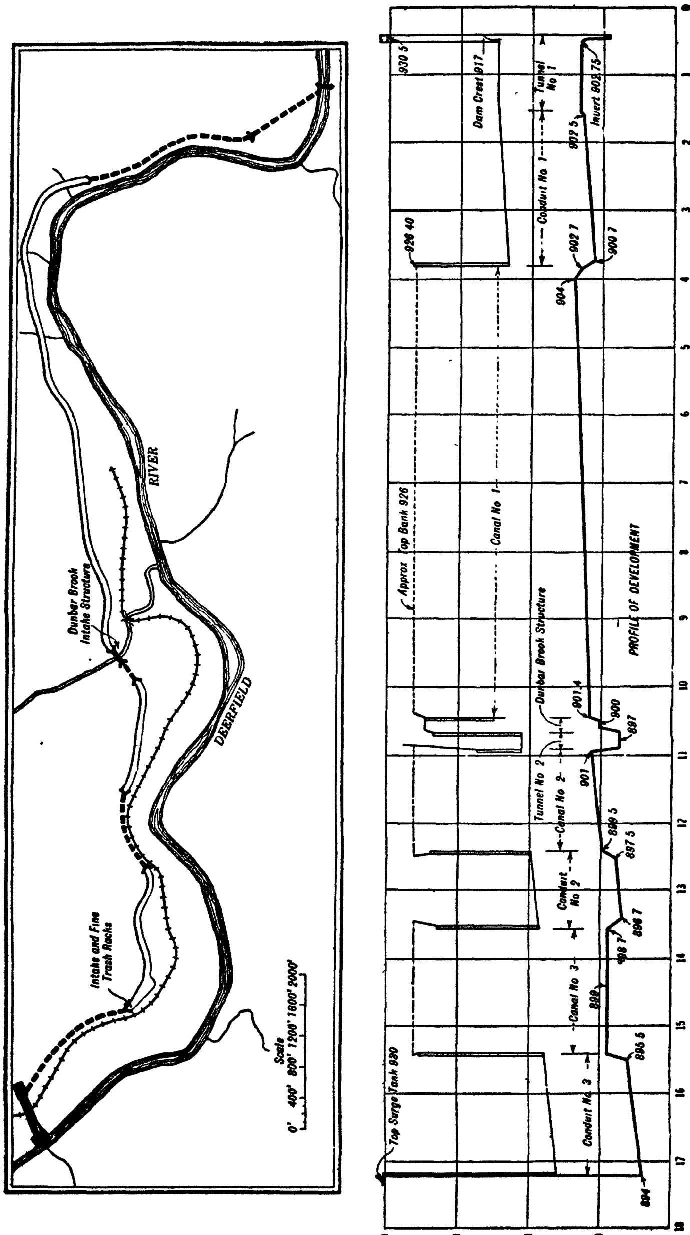


FIG. 106.—Hoosac Tunnel Plant No. 5, New England Power Co., Deerfield River, Mass. Three 7500 H.P. Units.

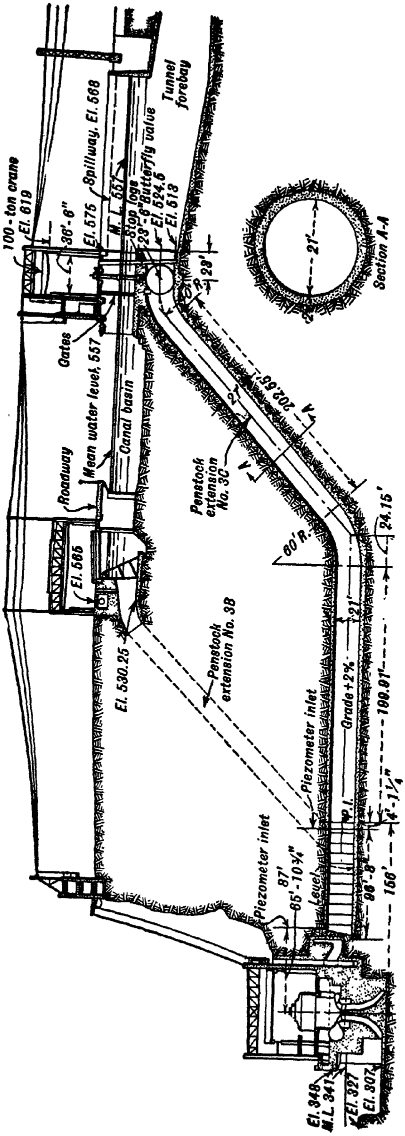


Fig. 107.—Cross-section through Cliff showing Relation of Penstocks, Intakes and Power House Station 3C, Niagara Falls Power Co. Eng. News-Record, Vol. 94, p. 305.

83. The Number of Units.—Factors affecting the capacity of the development are described in Chapter VI. The desired capacity having been fixed, the number of units in the plant next receives attention.

A few large units, as compared with a number of smaller units, is a dis-

tinct advantage as far as the cost of the development is concerned, because the cost per horse power decreases as the size of the unit increases. The required number of units depends upon the nature of the load and the number

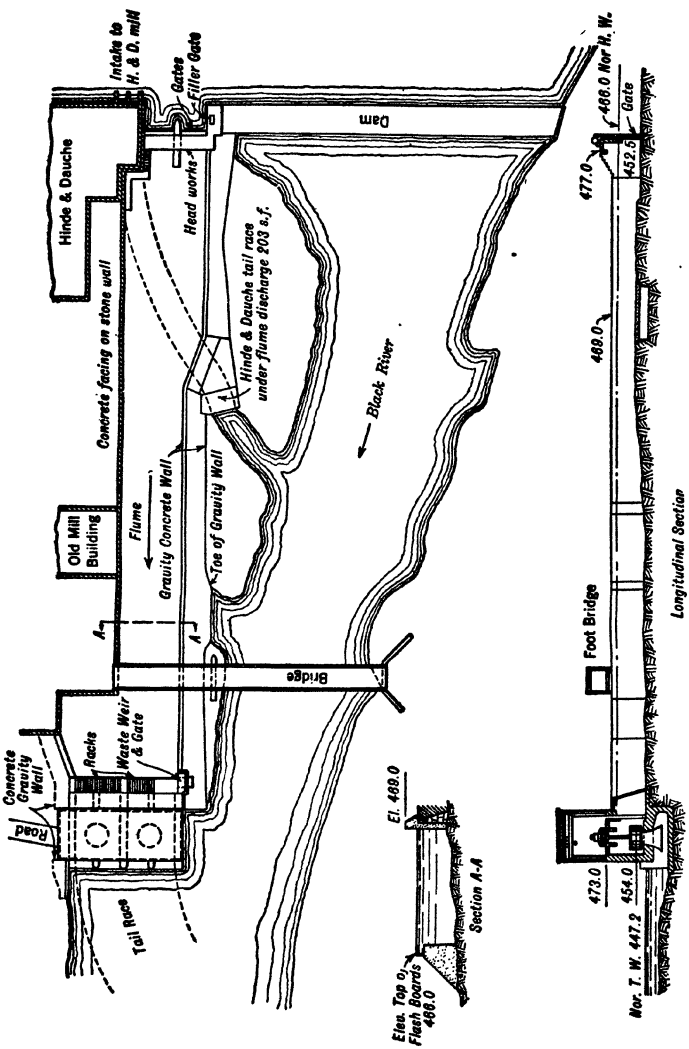


FIG. 108.—Sewall's Island Plant. Northern New York Utilities, Inc. 2 Units, 1250 H.P. Each.

of plants in the system. A development that is a part of an extremely large system can well afford to have only two units or even a single unit, because a unit out of commission in such cases would mean a very small percentage of

total power temporarily lost and, with a very large system, there is usually a steam auxiliary or other means of supplying the deficiency.

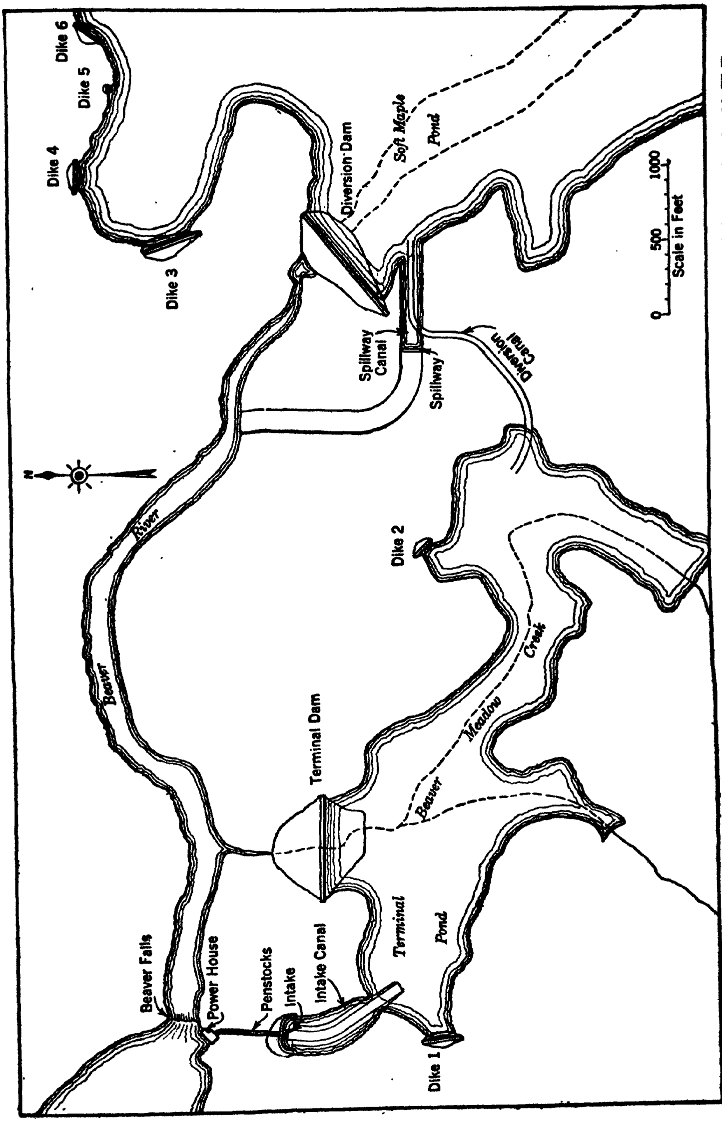


FIG. 109.—Soft Maple Development, near Croghan, N. Y. Northern New York Utilities, Inc. 2 Units, each 10,500 H.P. at 121.5 Ft. Head.

On the other hand, a development that alone serves a market cannot afford to rely on a single unit, for obvious reasons. Such developments, if

serving an important market without a steam plant standby, must have a spare unit for emergency; and the spare unit would, of course, be of the same size as the other units. To provide a spare unit for a single-unit plant doubles the installation. A spare unit for a two-unit plant adds 50 per cent and, for a three-unit plant adds 33⅓ per cent. Consequently, if a spare unit is required, the total installed capacity decreases as the number of units increases; but, since the cost per horse power increases as the number of units increases, there is for each case a definite and readily determined number of units that is most economical.

If the development is to serve a secondary-power market, a part of the

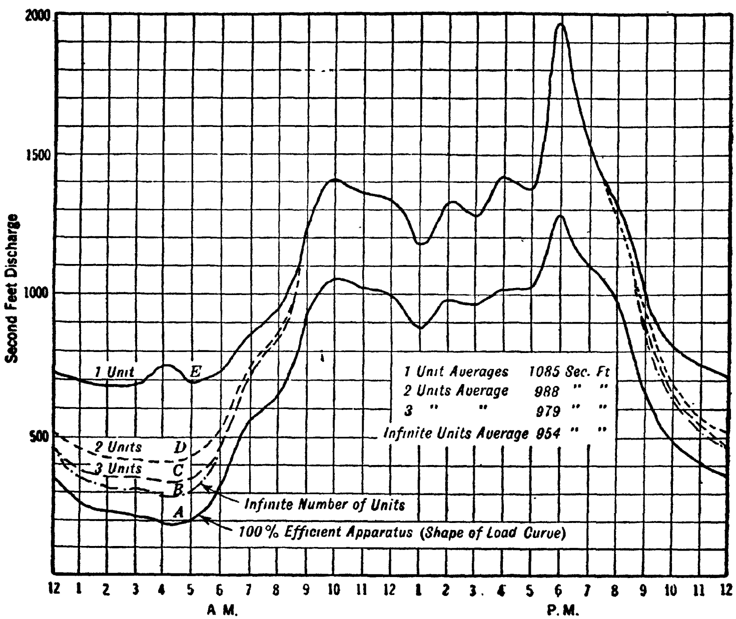


FIG. 110.

installation will be idle during periods of low water. At such times there is ample capacity, and during periods of abundant flow, a shut-down of a unit affects secondary power only. Therefore, a spare unit is not necessary in such cases.

Another point to be considered in fixing the number of units is the nature of the demand curve. If the market requires a single-unit plant to operate for considerable periods at an output equal to a small percentage of the peak, the average efficiency of the development is very low. Figure 41 shows the relative efficiency of one-, two- and three-unit plants, and Sec. 48 describes them. Figure 41 shows that a single-unit plant operating alone will have a much lower light-load efficiency than a plant having two or more units. The

average efficiency, of course, for a variable load, increases with the number of units.

Curve *A* of Fig. 110 shows the water required to deliver the power for the total-load curve of Fig. 35, based on 110-ft. net head and 100 per cent efficient apparatus. It is, therefore, truly symbolic of the shape of the load curve.

Curves *B*, *C*, *D*, and *E* of Fig. 110 show the water required for this load with an infinite number of units, three units, two units, and one unit, respectively, taking efficiencies from Fig. 41. The figure also shows very clearly the waste of water for small loads. It also shows that, for this typical load curve, small benefit accrues from the provision of more than two units. This feature is perhaps more clearly demonstrated by the following tabulation:

Units Used	Second-feet Average Discharge Required	Relative Average Discharge Required
1	1085	113.7
2	988	103 5
3	979	102.5
Infinite Number	954	100.0

The foregoing tabulation shows that, for this load curve, 10.2 per cent more water is required for one unit than for two units; but that only 1.0 per cent more water is required for two units than for three units.

From the engineer's standpoint there is no limit to the size of unit, but manufacturing, hauling, and shipping limitations fix the maximum capacity that can be adopted, and such limitations frequently require a number of units if the development is very large.

For plants that are a part of a large system, the effect of the load curve is not a governing feature if pondage is available, because all plants can be run at the point of best efficiency, irrespective of the number of units, simply by shifting the load from one plant to another and operating each plant at its point of best efficiency or not at all.

84. Consultation with Operators.—The operating department of the company should be freely consulted during all stages of the design and construction of the development. This is very essential for a number of reasons, among which may be mentioned the following:

(*a*) To obtain information as to standard details of other developments of the company, which affect the operating routine. Such details should be as similar as possible.

(*b*) To derive benefit from the experience of the operating department in the operation of similar plants on the same river or territory;

(*c*) To ascertain the general requirements of the system as a whole;

(*d*) To secure the benefit that the best-informed designing engineer can always derive from the advice of an experienced operator, no matter how lacking in technical knowledge the latter may be.

(*e*) To give the operator his inherent right to a voice in the selection of

apparatus and choice of details for a plant which he will be called upon to operate;

(*f*) To secure the good will of the operator. The measure of the value of a development, to those not familiar with its details, is the absence of criticism and the abundance of favorable comment. Human nature is everywhere the same, and it is not reasonable to assume that those who are required to operate a plant without being consulted in its design will exert great effort in advertising it favorably.

85. Velocities and Friction Heads.—A tabulation of a large number of modern plants discloses very little relation between the type of the development, the head developed, and the allowed friction losses. The judgment of designers and local conditions vary so widely that it is quite impossible to give, within very close limits, the practice in this respect, it being necessary to solve each problem according to the theory of economical design given in Sec. 87. It can be stated, however, that, other things being equal:

(*a*) The allowed friction loss at full load, and hence the velocities in the conduit system, increase as the head increases. Friction losses at full load vary in general between 5 and 10 per cent of the gross head, and average about 6 per cent. It is obvious that, for cases where full load or nearly full load is required for only a very short period each day, a larger loss can economically be allowed than if full load were of long duration.

(*b*) Full-load velocities at the racks are not usually affected by head losses, because the velocity is generally limited to that at which rakes can be easily used. Higher velocities can be used where mechanical rakes are installed than for hand operation. Velocities, at full load in the gross area of racks,[4] between 1.75 and 2.0 ft. per second are usually adopted, although slightly higher and lower velocities have been used. (See also Secs. 151 and 152.)

(*c*) Full-load velocities at intake gates vary between 2.5 and 8.0 ft. per second, with an average of about 4.0 or 5.0 ft. (See also Secs. 151 and 152.)

(*d*) Full-load velocities in high-line conduits usually decrease as the length increases, not only to avoid excessive friction losses but also to reduce the fluctuations of water surface at the surge regulator. Such velocities necessarily vary between wide limits. The velocities in high-line pipes, flumes, and tunnels usually vary between 5 and 15 ft. per second. No general statement can be made for velocities in canals, as too many conditions influence the choice.[5]

(*e*) Full-load velocities in penstocks for low and moderate-head plants are affected not only by considerations of economy but also by the factors of water-hammer and turbine speed regulation. Very high head plants are usually provided with automatic by-passes to pre-

[4] By "gross area" is meant the total-vertical area at the racks, including the area occupied by the racks and supports, but not the area occupied by concrete piers.

[5] See Chapter XVIII.

vent sudden stoppage of flow at times of turbine gate closure, and hence there is no definite limit to velocity other than that dictated by economy of design. Penstock velocities are seldom below 6.0 ft. per second and have been used as high as 20.0 ft. The average for moderate-head plants is about 8.0 ft.

(f) The generally adopted full-load velocity at the exit of the draft tube and in short tail races increases rapidly with the head, and usually varies between 3.0 ft. per second for very low heads and 8.0 ft. per second for the high heads.

86. Probability Curves.—The probable frequency of occurrence of future events, based on an extended record of past occurrences, can be estimated according to the law of probabilities. In connection with water powers, such studies are particularly useful in the determination of the probable frequency of the following events:

(a) Flood flows;
(b) Low river discharge;
(c) Depletion of storage reservoir;
(d) Low annual rainfall;
(e) High rates of rainfall.

An estimate of the probable frequency of flood flows of the Tennessee River at Chattanooga, Tenn., made according to the law of probabilities, is given below. The method is strictly applicable to all other similar problems.

Discharge records of the Tennessee River extend over a period of forty-one years, from 1875 to 1913, inclusive, 1916, and 1917. All floods [6] in excess of 100,000 sec.-ft. that occurred during that term of years are recorded in Col. 2 of Table XXVI, which indicates the number of occurrences of floods of a magnitude between the corresponding flood in Col. 1 and the one next below. In this case, 7 floods of a magnitude between 200,000 and 205,000 sec.-ft. occurred during the period of records.

In Col. 3 is given a summation of the occurrences indicated in Col. 2. Col. 3, therefore, shows the number of times, during the period, a given flood was equaled or exceeded. In this case a flood equal to or greater than 200,000 sec.-ft. occurred 37 times during the period.

According to the law of probabilities, the probable percentage of future floods that will equal or exceed a given flood, Q, may be obtained by the following equation:

$$p = \frac{100(n - 0.5)}{m}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (70)$$

where p = the probable percentage of future floods that will equal or exceed a given flood, Q, expressed as a whole number;

n = the number of times, during the period of records, a flood, Q, was equaled or exceeded as shown by Col. 3;

m = the total number of floods that occurred during the period of records, in this case being 179 floods.

[6] For this example, the basic-stage method of Sec. 33 is used.

TABLE XXVI

CALCULATIONS FOR PROBABILITY PLOTTING OF TENNESSEE RIVER

Q Peak Flow (1)	Number of Occurrences (2)	Summation of Occurrences (3)	Calculated Percentage p, of Future Floods (4)	Q Peak Flow (1)	Number of Occurrences (2)	Summation of Occurrences (3)	Calculated Percentage p, of Future Floods (4)
100,000	14	179	99.7	225,000	3	23	12.6
105,000	16	165	92.0	230,000	1	20	10.9
110,000	5	149	83.0	235,000	0	19	....
115,000	12	144	80.2	240,000	1	19	10.3
120,000	5	132	73.5	245,000	0	18	....
125,000	6	127	70.6	250,000	8	18	9.8
130,000	12	121	67.3	255,000	1	10	5.3
135,000	10	109	60.6	260,000	0	9	....
140,000	12	99	55.0	265,000	3	9	4.8
145,000	10	87	48.3	270,000	1	6	3.1
150,000	3	77	42.7	275,000	0	5	....
155,000	5	74	41.0	280,000	2	5	2.0
160,000	7	69	38.2	285,000	0	3	....
165,000	3	62	34.4	305,000	1	3	1.4
170,000	4	59	32.7	310,000	0	2	....
175,000	3	55	30.4	345,000	1	2	0.84
180,000	3	52	28.8	350,000	0	1	....
185,000	2	49	27.1	360,000	1	1	0.28
190,000	7	47	26.0	365,000	0	0	....
195,000	3	40	22.1				
200,000	7	37	20.4		179		
205,000	0	30	.....				
210,000	1	30	16.5				
215,000	4	29	15.9				
220,000	2	25	13.7				

This equation gives the values of p in Col. 4, which indicates that 20.4 per cent of all future floods above 100,000 sec.-ft. probably will equal or exceed 200,000 sec.-ft.

Values of flood discharge from Col. 1 are plotted as ordinates, and percentages from Col. 4 as abscissae, on probability paper,[7] as indicated in Fig. 111. In many cases it is desirable to estimate the probable percentage of floods of a magnitude in excess of any flood of record. This may be done by extending a line drawn through the plotted points. The line may be slightly curved in some instances, making the extension subject to greater error.

[7] Probability paper was first devised by Allen Hazen in 1913 (See Trans. Am. Soc. C. E., Dec., 1914, p. 1539). The spacing of the vertical lines was computed by Hazen from figures taken from probability curve tables and arranged so that the line that represents the probability curve, when plotted on it, is straight. If the data for any series correspond strictly to the normal law of error, the points plotted on this paper with arithmetic ordinates will all be in a straight line. In the normal law of error, the probabilities above the average are the same as those below the average; but, in most of the practical cases in connection with water powers, this relation does not hold, the lower limit of the Tennessee flood problem being 100,000 sec.-ft. and the upper limit infinity. Probability paper is published by the Codex Book Co. of New York City and can be obtained with either arithmetic and logarithmic ordinates. Either arithmetic or logarithmic ordinates may be used, whichever gives the best curve for extrapolation.

Fig. 111 indicates that only 0.1 per cent of future floods may equal or exceed 414,000 sec.-ft. In many cases, particularly when the record is comparatively short, or when the probability line is curved, it is difficult to project the line to floods greater than those of record, and the Foster method of deriving the probability line may be used, as follows:

H. Alden Foster [8] has derived mathematically a method by which the probability curve can be obtained directly from the fundamental data.

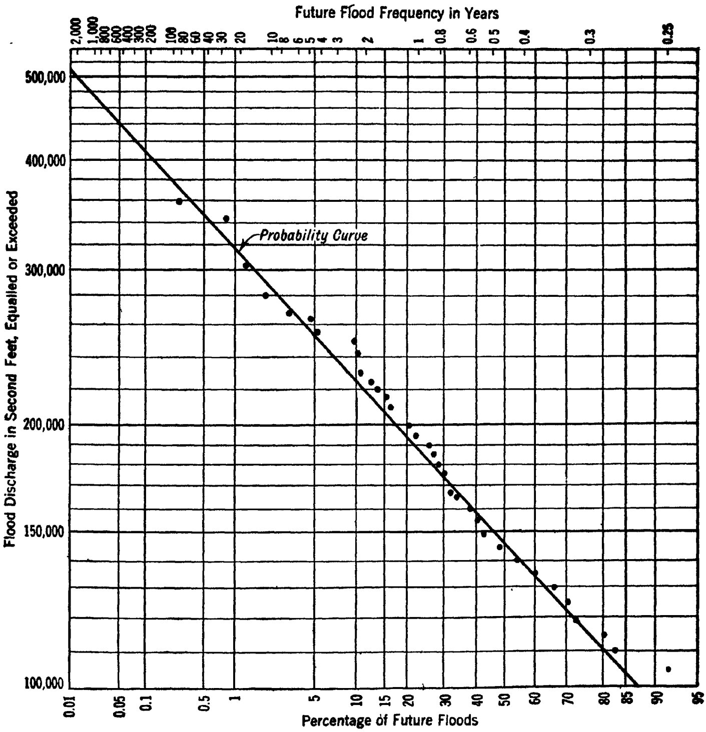


FIG. 111.—Probability Curve of Tennessee River at Chattanooga, Tenn.

This method is shown here for the case in question and is similar for all other cases.

In Table XXVII, Cols. 1 and 2 are identical with Cols. 1 and 2 of Table XXVI. The first step is to find the mean flood. As the summation of all floods in Col. 3 (product of Cols. 1 and 2) is 28,037,000 and the total number of floods is 179, the mean flood is:

$$\text{Mean flood} = \frac{28{,}037{,}000}{179} = 156{,}500 \text{ sec.-ft.}$$

[8] See Reference 8, Sec. 88.

TABLE XXVII

CALCULATIONS FOR PROBABILITY CURVE OF TENNESSEE RIVER (FIRST STEP)

Q, Peak Flow	Number of Occurrences	Summation of Floods	Flood in Terms of Mean Flood	Variation from Mean = V		V^2	ΣV^2	V^3		ΣV^3	
				+	−			+	−	+	−
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
100,000	14	1,400,000	0.638	....	0.362	0.1310	1.835	....	0.0475	....	0.6650
105,000	16	1,680,000	0.671	....	0.329	0.1082	1.732	....	0.0353	....	0.5650
110,000	5	550,000	0.703	....	0.297	0.0888	0.444	....	0.0265	....	0.1325
115,000	12	1,380,000	0.735	....	0.265	0.0702	0.842	....	0.0186	....	0.2232
120,000	5	600,000	0.767	....	0.233	0.0542	0.271	....	0.0126	....	0.0630
125,000	6	750,000	0.799	....	0.201	0.0402	0.241	....	0.0081	....	0.0487
130,000	12	1,560,000	0.831	....	0.169	0.0275	0.330	....	0.0048	....	0.0576
135,000	10	1,350,000	0.863	....	0.137	0.0187	0.187	....	0.0026	....	0.0257
140,000	12	1,680,000	0.895	....	0.105	0.0110	0.132	....	0.0012	....	0.0139
145,000	10	1,450,000	0.927	....	0.073	0.0052	0.052	....	0.0004	....	0.0039
150,000	3	450,000	0.959	....	0.041	0.0016	0.005	....	0.0001	....	0.0002
155,000	5	775,000	0.991	....	0.009	0.0001	0.000	....	0.0000	....	0.0000
160,000	7	1,120,000	1.022	0.022	....	0.0005	0.003	0.0000	....	0.0000	
165,000	3	495,000	1.054	0.054	....	0.0029	0.009	0.0002	....	0.0006	
170,000	4	680,000	1.087	0.087	....	0.0076	0.030	0.0007	....	0.0028	
175,000	3	525,000	1.118	0.118	....	0.0139	0.042	0.0016	....	0.0048	
180,000	3	540,000	1.150	0.150	....	0.0225	0.067	0.0035	....	0.0105	
185,000	2	370,000	1.182	0.182	....	0.0331	0.066	0.0060	....	0.0120	
190,000	7	1,152,000	1.214	0.214	....	0.0458	0.321	0.0097	....	0.0679	
195,000	3	585,000	1.246	0.246	....	0.0604	0.181	0.0147	....	0.0441	
200,000	7	1,400,000	1.278	0.278	....	0.0772	0.540	0.0215	....	0.1505	
205,000	0	0									
210,000	1	210,000	1.342	0.342	....	0.117	0.117	0.0400	....	0.0400	
215,000	4	860,000	1.374	0.374	....	0.140	0.559	0.0523	....	0.2092	

220,000	2	440,000	1.406	0.406	....	0.165	0.329	0.0670	....	0.1340	
225,000	3	675,000	1.438	0.438	....	0.192	0.575	0.0841	....	0.2523	
230,000	1	230,000	1.470	0.470	....	0.221	0.221	0.1038	....	0.1038	
235,000	0	0									
240,000	1	240,000	1.533	0.533	....	0.284	0.284	0.1512	....	0.1512	
245,000	0	0									
250,000	8	2,000,000	1.597	0.597	....	0.356	2.848	0.2125	....	1.7000	
255,000	1	255,000	1.630	0.630	....	0.397	0.397	0.2500	....	0.2500	
260,000	0	0									
265,000	3	795,000	1.692	0.692	....	0.478	1.434	0.3310	....	0.9930	
270,000	1	270,000	1.725	0.725	....	0.525	0.525	0.3810	....	0.3810	
275,000	0	0									
280,000	2	560,000	1.769	0.769	....	0.592	1.184	0.4540	....	0.9080	
285,000	0	0									
305,000	1	305,000	1.948	0.948	....	0.897	0.897	0.8520	....	0.8520	
310,000	0	0									
345,000	1	345,000	2.204	1.204	....	1.450	1.450	1.7430	....	1.7430	
350,000	0	0									
360,000	1	360,000	2.300	1.300	....	1.690	1.690	2.2000	....	2.2000	
365,000	0	0									
	179	28,037,000					+19.840			+10.2107	−1.7987

Column 4 gives the flood in terms of the mean flood and is derived as follows:

$$\text{Items of Col. 4} = \frac{\text{Corresponding item in Col. 1}}{156{,}500}$$

Columns 5 and 6 give the value V, or the variation from the mean, and are equal to the corresponding items in Col. 4, less unity.

Column 7 gives the square of the corresponding items in Cols. 5 and 6. Col. 8 gives the summation of Col. 7 and is equal to the product of the corresponding items in Cols. 2 and 7.

Columns 9 and 10 give the cube of the corresponding items in Cols. 5 and 6. Cols. 11 and 12 give the summation of Cols. 9 and 10 and are equal to the corresponding items in Col. 2 times items in Cols. 9 and 10.

Then

$$\Sigma V^3 = 10.2107 - 1.7987 = 8.412;$$
$$\Sigma V^2 = 19.84;$$
$$m = \text{number of occurrences (Col. 2)} = 179;$$

$$\text{Coefficient of variation}^{9} = cv = \sqrt{\frac{\Sigma V^2}{m-1}} = \sqrt{\frac{19.84}{179-1}} = 0.334.$$

$$\text{Coefficient of skew}^{9} = cs = \frac{\Sigma V^3}{(m-1)(cv)^3} = \frac{8.412}{(179-1)(0.334)^3} = 1.268.$$

The Coefficients of Variation and Skew having been derived, Mr. Foster's tabulation (reproduced here in Table XXVIII) gives the frequency variation in terms of the Coefficients of Variation and Skew. The use of this table is demonstrated for this example in Table XXIX.

The results of Table XXIX have been plotted in Fig. 111 and the resulting curve has been extended to the margin of the figure. In this case it is nearly straight; but in many cases it may be decidedly curved.

It is not claimed that Foster's mathematical analysis, previously explained gives precise results, inasmuch as the calculations must be based, as for other methods, upon limited data. It is well, therefore, to compute Table XXVI and plot the resulting points as in Fig. 111 for all cases, using Foster's method to assist in the determination of the actual probability curve. One will afford a check on the other.

In order to obtain the frequency of occurrence of future floods, let y equal the number of years of records. Then the average frequency of floods during the period is:

$$f = \frac{m}{y} \text{ floods per year.}$$

[9] Foster states that, for floods, the coefficient of variation can be obtained with considerable accuracy from a record of moderate length but the coefficient of skew cannot be determined accurately except for records of considerable length. He states that, in all cases, no matter what the computed coefficient of skew is, it must be adopted at least equal to twice the coefficient of variation. This was determined from investigation of a number of streams.

TABLE XXVIII*

RUNOFF VARIATION IN TERMS OF COEFFICIENT OF VARIATION

(Skew-curve Factors to be Multiplied by Coefficient of Variation and Added to or Subtracted from Mean)

Coefficient of Skew	Terms above Mean in Percentage	Variation from Mean, in Terms of C. V. Percentage of Time =														
		0.1	1.0	5	10	20	30	40	50	60	70	80	90	95	99.0	99.9
0.0	50.0	+3.09	+2.33	+1.64	+1.28	+0.84	+0.52	+0.25	0.00	−0.25	−0.52	−0.84	−1.28	−1.64	−2.33	−3.09
0.2	48.7	3.38	2.48	1.69	1.30	0.83	0.51	0.22	−0.03	0.28	0.55	0.85	1.25	1.58	2.18	2.81
0.4	47.3	3.67	2.62	1.74	1.32	0.82	0.48	0.19	0.06	0.31	0.57	0.85	1.22	1.51	2.03	2.54
0.6	46.0	3.96	2.77	1.79	1.33	0.80	0.45	0.15	0.09	0.34	0.58	0.86	1.19	1.45	1.88	2.28
0.8	44.7	4.25	2.90	1.83	1.34	0.78	0.42	0.12	0.13	0.37	0.60	0.86	1.16	1.38	1.74	2.03
1.0	43.3	4.54	3.03	1.87	1.34	0.76	0.38	0.08	0.16	0.40	0.61	0.86	1.12	1.31	1.59	1.80
1.2	42.0	4.82	3.15	1.90	1.35	0.74	0.35	0.05	0.19	0.42	0.62	0.85	1.08	1.25	1.45	1.59
1.4	40.7	5.11	3.28	1.93	1.34	0.71	0.32	+0.02	0.22	0.44	0.63	0.84	1.05	1.18	1.32	1.40
1.6	39.4	5.39	3.40	1.96	1.33	0.68	0.28	−0.01	0.25	0.46	0.64	0.82	1.00	1.11	1.19	1.24
1.8	38.1	5.66	3.50	1.98	1.32	0.64	0.24	0.05	0.28	0.48	0.64	0.80	0.95	1.03	1.08	1.11
2.0	36.8	5.91	3.60	2.00	1.30	0.61	0.20	0.08	0.31	0.49	0.64	0.78	0.89	0.95	0.99	1.00
2.2	35.5	6.20	3.70	2.01	1.28	0.58	0.17	0.11	0.33	0.49	0.63	0.75	0.84	0.89	0.90	0.91
2.4	34.3	6.47	3.78	2.01	1.25	0.54	0.13	0.14	0.35	0.50	0.62	0.71	0.79	0.82	0.83	0.83
2.6	33.0	6.73	3.87	2.01	1.23	0.51	0.10	0.17	0.37	0.50	0.60	0.68	0.74	0.76	0.77	0.77
2.8	31.9	6.99	3.95	2.02	1.20	0.47	0.06	0.20	0.38	0.50	0.59	0.65	0.70	0.71	0.71	0.71
3.0	30 8	+7.25	+4.02	+2.02	+1.18	+0.42	+0.03	−0.23	−0.40	−0.50	−0.57	−0.62	−0.65	−0.66	−0.67	−0.67

* From Trans. Am. Soc. C. E., Vol. LXXXVII, 1924, p. 162.

TABLE XXIX

CALCULATIONS FOR PROBABILITY CURVE OF TENNESSEE RIVER (SECOND STEP)

Percentage of Time	0.1	1	5	20	40	80	95	99.9
Variation * in terms of cv, from Table XXVIII	+4.92	+3.20	+1.91	+0.73	+0.04	−0.85	−1.22	−1.51
Variation † for $cv=0.334$	+1.645	+1.070	+0.637	+0.244	+0.0134	−0.284	−0.408	−0.505
Flood in terms of mean ‡	2.645	2.070	1.637	1.244	1.0134	0.716	0.592	0.495
Flood §	414,000	324,000	256,000	195,000	158,000	112,000	92,700	77,500

* Interpolated between coefficients of skew of 1.2 and 1.4 for the derived skew of 1.268.
† Product of 0.334 and line next above.
‡ Unity plus line next above.
§ Product of 156,500 with line next above.

Therefore, any period of years, Y, would experience:

$$N = fY = \frac{mY}{y} \text{ floods.}$$

If p per cent of these floods equaled or exceeded a given flood, Q, as indicated in Col. 4 of Table XXVI, the number of floods equaling or exceeding flood Q during the period Y, is,

$$N' = pN = \frac{mpY}{100y} \text{ floods.}$$

And the interval, in years, between floods equaling or exceeding Q is:

$$I = \frac{Y}{N'} = \frac{100y}{mp}. \qquad (71)$$

In this example:

$$I = \frac{100 \times 41}{179p} = \frac{22.9}{p}.$$

By means of this equation, the upper scale of flood frequency in Fig. 111 is calculated from the lower scale of percentage. The probable frequency of future floods can now be taken from Fig. 111, which indicates that a flood equal to or greater than 188,000 sec.-ft. may be expected once each year; that a flood equal to a greater than 380,000 sec.-ft. may be expected once every 100 years; and a flood of 502,000 sec.-ft. once every 2000 years.

The greater the number of events during the period of records, the more uniform will be the plotted points and the greater the degree of accuracy of the projected probability curve. It will be noticed, in most cases, that the extremes of the plotted points are less indicative of the trend of the probability curve. This is to be expected because there are a far greater number of events of medium magnitude.

Great care should be exercised in the use of probability curves, particularly if the number of recorded events is not great. The frequency and magnitude of events during a relatively short period may correspond to a low or a high cycle, and therefore may not be indicative of average conditions. In studies involving climatological phenomena, it is desirable to have at least twenty-five

or thirty years of records for best results (see Table XXVI); but the use of shorter periods is much to be preferred to the application of a factor of safety of 1.5 or 2.0 to the governing occurrence of past records, as has often been done. Considering only the controlling condition in a series, with no weight given to the periodic occurrence of lesser conditions, makes the result purely a matter of chance, depending upon what may have been the controlling condition. By studying the conditions in the whole series, by means of a probability curve, one brings into the problem the frequency of lesser conditions, which indicate to a much greater degree of accuracy the probable frequency of future events.

Estimates using probability curves to indicate the probable frequency of extraordinary conditions should be backed up with data available from all other sources. It should always be borne in mind that, in such cases, we are dealing in probabilities and that the result of the studies indicates only the average of future occurrences. Thus, a flood that may be exceeded, on an average, only once in a given period of years, may occur several times in that period.

87. Theory of Economical Design.—For the purpose of making studies of economical design, the different structures of a development may be divided into two classes:

(1) Those structures that are designed for a fixed duty, and influence in no appreciable way the energy output of the development. In this class would be placed such items as superstructure, highways, dams of a fixed height, and similar structures. The most economical design for structures of this class is that which, for the given duty, will require the least annual operating, depreciation, and interest charges. The laws of economical design are the same as those for structures in general, and will not be treated here.

(2) Those structures, the design of which influences not only the operating, depreciation, and interest charges, but also the energy output of the development and hence the annual income. In this class are included pipe lines, the diameter of which affects the energy lost in friction, dams of possible variation in height, the height of which affects the head available for power, canals, transmission-line copper, etc. This class of structures, being governed by extraordinary considerations and requiring special analysis, is the subject of the following discussion.

The following theory of economical design is based upon the premise that a portion of the necessary lost energy in the structure may be reclaimed at an increase in cost. Thus, in a pipe line, the friction head may be reduced by an increase in diameter and, in the case of a dam, the head available for power may be increased by increasing the height of the dam.

There are three distinct hypotheses upon which the theory of economical design may be based.

(1) Funds for investment in the project, because of financial stringency, are limited to the amount necessary for the success of the project.

(2) Funds for investment in the project are limited only by the necessity for making the project as attractive as possible.
(3) Funds for investment are unlimited, provided that the return on each dollar invested will equal or exceed a fixed minimum.

Figure 112, for a pipe-line study, is shown as an exaggerated typical example, to explain the conditions governing the three designing hypothesis.

Curve *A* shows the estimated per cent net return on a complete development, corresponding to different adopted pipe-line diameters. The maximum return of 20 per cent is seen to correspond to a pipe-line diameter of 100 in.

Curve *B* shows the estimated per cent net return on increments of investment for small increases in pipe-line diameter. A small expenditure necessary to increase the diameter from 110 in. to a slightly larger diameter would, because of the resulting increased output, return 15 per cent.

Assume, for this example, that a net return of at least 15 per cent is required on all money invested in the project.

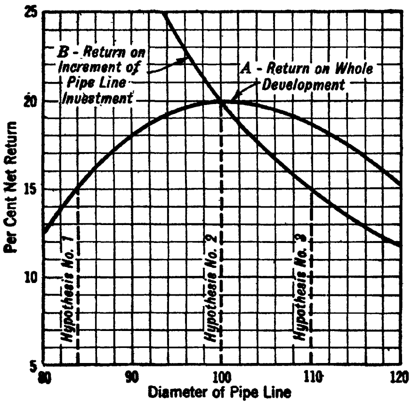


FIG. 112.

Under the first hypothesis, the economical diameter of the pipe would be 84 in., as indicated in the figure, corresponding to a satisfactory return of 15 per cent. Funds being limited to the amount required for the success of the project, additional investments to increase the diameter of the pipe would not be made, although, if made, they would be very profitable. Fortunately, this hypothesis represents a condition that is seldom specified for the design.

Under the second hypothesis, the economical diameter of the pipe would be 100 in., because this corresponds to the maximum possible per cent net return on the whole development. This hypothesis should be adopted when it is desired to make the project as attractive as possible from an investment standpoint.

Under the third hypothesis, the pipe line would be 110 in. in diameter, because, funds being unlimited, the pipe would be enlarged in diameter until the last increment of investment for that purpose resulted in a return exactly equal to the 15 per cent minimum desired net return on money invested. This hypothesis is used for final designs when the greatest possible output from the development, consistent with satisfactory returns on investments, is desired. It is also frequently used in reports designed to show the largest possible value of the water rights as, obviously, any return on investments in the pipe line, in excess of the minimum requirement, will leave a surplus which can be capitalized and added to the value of the rights.

As mentioned heretofore, the first hypothesis is quite unusual and will not be discussed further. The second and third hypotheses are quite similar as to procedure and may be expressed as follows:

Rule 1 for Economical Design.—**A particular design is most economical when an infinitely small additional investment, to increase the energy output, will give a per cent net return exactly equal to the minimum desired net return on money invested.**

The only difference in the application of this rule for the second and third hypotheses is in the interpretation of the words "the minimum desired net return on money invested."

For the second hypothesis, this should be interpreted to mean a return equal to that on the whole development, it being noted from Fig. 112 that curves A and B intersect when the return on the whole development is a maximum.

For the third hypothesis, "minimum desired net return on money invested" should be interpreted to mean the minimum limit fixed by investors.

If these interpretations are borne in mind, the procedure is exactly the same for the two hypotheses.

Unfortunately, the per cent net return on the whole development, for the second hypothesis, is not exactly known until the last item of design is completed. Hence, for this hypothesis, it is necessary to assume, tentatively, the return on the whole development, and to design the different structures on that basis. Then, when the last structure is designed, if the return on the whole development is different from that assumed, the process must be repeated.

This method, therefore, is not subject to exact procedure. To obtain an exact result would be an endless task, as there are so many items in the development that permit of economic considerations, the pipe line alone requiring special calculations for each section subject to different heads or other conditions. Consequently, the per cent return on the whole development, indicated by a preliminary tentative design, is usually adopted as the criterion, and the calculations are not repeated unless the final complete design shows a return quite different from that assumed. As a matter of fact, the whole theory of economical design of any structure must be based on assumptions, many of which are not known exactly, while others may be altered, in the course of a few years, by changing market conditions. It is not customary, therefore, to be too precise in making determinations of this character, unless the item in question is of sufficient importance or the data so definite as to justify a considerable amount of study.

The theory will now be explained in detail for a pipe line, although it is equally applicable to any other structure, the details of which affect the energy output.

In pipe lines of some length, the internal pressure and other conditions affecting the cost vary greatly at different parts. Hence, it is customary to make separate calculations for a number of short lengths, average conditions being used for each length. A length of 100 ft. is used in the following example:

In Table XXX, the construction cost of Line 2 includes that of the pipe,

saddles, piers, excavation, and all other items affected by a change in diameter. Actually, the cost of the surge tank is affected by the diameter of the pipe; but, unless the conditions are such that the pipe can be considered as a whole instead of in short lengths as explained heretofore, it is not practicable to include the cost of the surge tank in the investigation.[10]

The annual operating charges of Line 3 include maintenance, repairs, taxes, depreciation, and all other annual charges affected by the diameter, except interest and profit.

The loss of head in Line 4 is the loss for the average discharge.

Lost energy, in Line 5, is from Eq. (18) of Sec. 50, CH_f being used for the lost head instead of $H - CH_f$, and 8760 hours in one year substituted for T, or:

$$\text{Line 5} = 742QCH_fe.^{11}$$

Now, assuming $Q = 200$, $C = 2.05$, and $e = 0.75$:

$$\text{Line 5} = 228{,}000\,H_f = 228{,}000 \text{ times Line 4.}$$

Line 6 is the value of lost energy in Line 5. The kilowatt-hours are multiplied by the unit value of energy at the point of measurement corresponding to the efficiency, e, used in the foregoing equation. The point of measurement may be at the customers' meters, at the power house, or even at the pipe

TABLE XXX

FOR 100-FT. LENGTH OF PIPE

1. Diameter of pipe, in feet	6.5	7.0	7.5	8.0	8.5	9.0
2. Construction cost	$2411.00	$2627.00	$2849.00	$3076.00	$3324.00	$3582.00
3. Annual operating charges	$75.16	$81.94	$88.90	$96.04	$103.89	$112.10
4. Loss of head, in feet	0.133	0.090	0.065	0.048	0.036	0.027
5. Annual lost energy in kw.-hr	30,310	20,530	14,810	10,940	8,200	6,160
6. Value of energy lost at 7 mills per kw.-hr	$212.20	$143.70	$103.70	$76.60	$57.40	$43.10
7. Increment of increased gross return for each 0.5 ft. increase in diameter (from Line 6)	....	$68.50	$40.00	$27.10	$19.20	$14.30
8. Increment of increased operating charges for same conditions (from Line 3)	......	$6.78	$6.96	$7.14	$7.85	$8.21
9. Increment of increased net return for same conditions (from Lines 7 and 8)	....	$61.72	$33.04	$19.96	$11.35	$6.09
10. Increment of construction cost for same conditions (from Line 2)	....	$216.00	$222.00	$227.00	$240.00	$258.00
11. Per cent net return on increment investment (from Lines 9 and 10)	....	28.56	14.87	8.79	4.58	2.36

[10] See further discussion of this feature in Sec. 183.

[11] CH_f is the lost productive head as explained in Sec. 50.

line, provided the proper value of its worth and the proper efficiency are used. In other words, Line 6 is intended to show the actual loss of income due to lost head, and it may be obtained by any means at the engineer's disposal.

Lines 7 to 11, inclusive, are explained in the table.

The 28.56 per cent return on the investment in Line 11, to increase the diameter from 6.5 to 7.0 ft., is the average rate of return for all infinitesimal increases between 6.5 and 7.0, and is approximately the actual rate of return for a small increase at 6.75-ft. diameter, midway between. Therefore, each rate of return in Line 11 is stepped back, in this way, 0.25 ft. and plotted as Curve A of Fig. 113. With this curve may be obtained the most economical diameter for any desired per cent net return on the money invested, that for a return of 15 per cent being 7.23 ft.

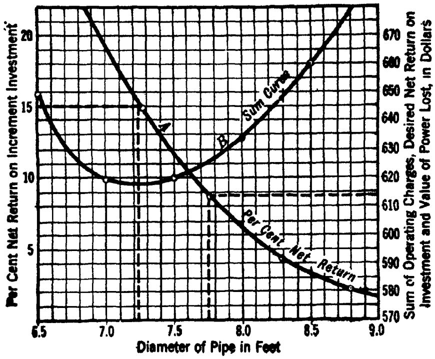


FIG. 113.

The foregoing method may be used for the determination of the most economical size of any other structure. As applied to a dam, for instance, Line 1 would show varying heights of the dam; Line 2 would include, not only the cost of the dam, but also the increased cost of the pipe line and other items affected by raising the dam; and Lines 4, 5, and 6, resulting in Line 7 for increased gross return, would be superseded by another obvious method of determining the increased revenue accruing from the resulting increased head.

Rule 1 for economical design, when applied to pipes, gates, canals, and other structures in which increased output can be obtained by a decrease in friction, may be put in a different form, known as Adam's Theorem.

Let C = total cost of a structure of a given size;
(dC) = infinitesimal increase in cost for an infinitesimal increase in size or other means of reducing friction;
$-P$ = total energy lost in the structure by friction, for the given size;
$-(dP)$ = infinitesimal increase in energy due to the investment (dC);
b = unit value of energy sold;
O = total annual operating charges for the structure of the given size;
(dO) = infinitesimal increase in operating charges for the infinitesimal change in size; and
R = desired per cent net return on money invested.

From Rule 1:

$$-b(dP) - (dO) = R(dC).$$

Transposing:

$$(dO) + R(dC) + b(dP) = 0.$$

But this is a criterion for a minimum; therefore, integrating, we have:

$$O + RC + bP = \text{A minimum.} \quad \quad (72)$$

From this expression we can now write Adam's Theorem as follows:

Rule 2 for Economical Design.—**A particular design is most economical when the sum of the total annual operating charges, the desired net return on the investment, and the value of energy lost therein is a minimum.**

The use of this rule for the preceding example is shown in Table XXXI in which the three factors of Rule 2 are tabulated. The desired net income is based on 15 per cent return. The sum of the three factors is plotted as Curve B in Fig. 113 opposite the corresponding diameters, and it is found that the minimum occurs for 7.23-ft. diameter, as derived also by Rule 1.

TABLE XXXI

(Line References are for Preceding Table)

Diameter of pipe in feet	6.5	7.0	7.5	8.0	8.5	9.0
Annual operating charges (Line 3)	$75.16	$81.94	$88.90	$96.04	$103.89	$112.10
Desired annual net return (15 per cent of Line 2)	361.65	394.05	427.35	461.40	498.60	537.30
Annual value of energy lost (Line 6)	212.20	143.70	103.70	76.60	57.40	43.10
Sum	$649.01	$619.69	$619.95	$634.04	$659.89	$692.50

In some cases there may be a mathematical relation between the three factors of Rule 2, and a general equation may be written for the determination of the most economical size. Where this is possible, it has been treated herein under the discussion of the particular structure.

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9. Many of the references of Section 38 (Bibliography, Flood Flows) contain valuable matter relating to the theory of probability curves. Other references on this subject are listed above under 8 and 9.

CHAPTER XI

TIMBER DAMS

By Joel D. Justin

89. Advantages of Timber Dams.—Timber dams were formerly much used in this country, and are still built in sections where transportation is difficult and timber plentiful. Under such circumstances they are entitled to serious consideration as a competitor of the concrete dam.

The life of a well-built timber dam has been variously estimated at from twenty to thirty years. However, in this, as in many similar cases, it is difficult to estimate the life of a structure that is properly maintained. Dams reputed to be 80 to 100 years of age have been cited; but, in such instances, probably a very small percentage of the original timber remained.

The maintenance charges for timber dams are large, particularly at sites where large floods and ice runs are frequent. Leakage is frequently very great, and the leaks are often exceedingly difficult to repair if the dam is relatively high and if a drawdown of the pond for repairs seriously affects operation. The large maintenance charges and leakage have created a prejudice against this type of dam. Hence, even in sections where virgin timber is plentiful, the use of timber dams to-day is rather the exception.

However, timber dams are frequently built at considerably less first cost than concrete dams, and are often adopted for this reason when money is scarce. As the maintenance and depreciation charges on concrete dams are practically negligible, the interest charges on a concrete dam theoretically, must be less than the sum of the interest, maintenance, and depreciation charges on a timber dam to warrant the adoption of the former type. However, there are doubtless many instances in which a concrete dam has been constructed although true economy would have dictated the selection of a timber structure.

90. The A-frame Type.—In Fig. 114 is shown a type of timber dam known as the A-frame type. It is generally built of squared timbers and planks and is not rock-filled. For its stability it depends on the weight of water on its deck and the anchorage of the sills to the foundation. It is probably the ancestor of the reinforced, flat-deck, hollow type of concrete dam. The deck makes an angle of 30 degrees or less with the horizontal.

The sills, *a*, are first fastened to the ledge rock by wedge bolts or anchor bolts, preferably grouted in. The struts, *b*, are then framed to the sills and held in place by cross bracing and batten blocks. The wales, *c*, are then placed, the entire structure being thoroughly drift-pinned together. These

bents are placed from 6 to 12 ft. apart according to the height of the dam and the size of timbers used. Across the bents are placed the studs, *d*, to which the lagging, *e*, is nailed. The lagging should be either tongued and grooved or lapped, and should not be less than 2-in. stuff.

91. The Rock-filled Crib Type.—In this type of timber dam, cribs of

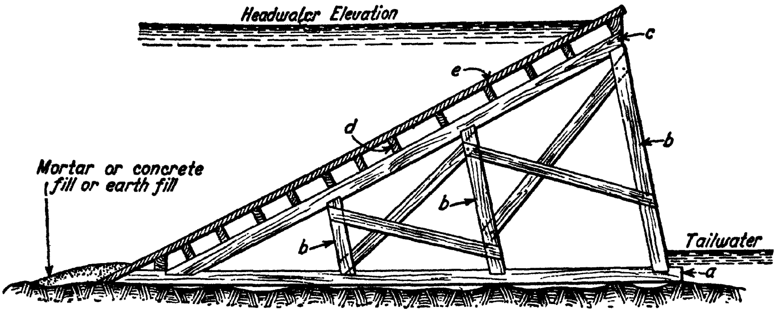

FIG. 114.—"A" Frame Type of Timber Dam.

round or squared timbers are drift-bolted together, filled with rock fragments or boulders, and topped by a plank deck. The timbers are usually spaced about 8 ft. centers both ways. The bottom timbers of the cribs are often pinned to the rock foundation if the site is not submerged. Fig. 115 shows a typical dam of this kind; but many different forms have been adopted.

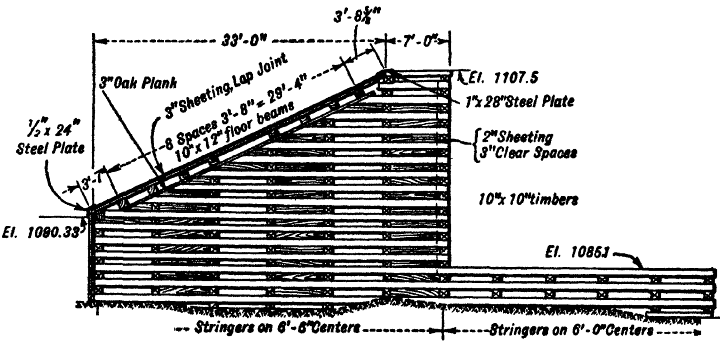

FIG. 115.—Rock-filled Timber-crib Dam at Ocoee, Tennessee.

For rock foundations, where erosion from the overflow would not be serious, the form of section is frequently made to resemble that of the A-frame type, in order to take advantage of the weight of the water on the sloping deck. For low dams on soft foundations, this section is usually reversed, having a nearly vertical upstream face and a long, sloping downstream face, frequently stepped, in order to drop the water without great disturb-

ance. Between these extremes many shapes of section have been adopted, some having both upstream and downstream faces sloping or stepped as in Fig. 117.

92. The Beaver Type.—Another type of timber dam, which is used infrequently and only for low heads, is the beaver-type dam. Round timbers are used for the bents as in Fig. 116. The upstream slope of such a dam should not be steeper than 1 on 2. The butts of the timbers all point downstream. Between the butts are placed spacer logs, which are drift-pinned to the other logs. Also, the tips of the timbers pointing upstream are drift-pinned together and the bottom timbers are fastened to the foundation with anchor bolts, if possible. There is usually a plank deck. Sometimes a mat of brush, or the branches of the trees used, take the place of the plank deck.

93. Stability of Timber Dams.—The theory of design of masonry dams, given in Chapter XII, is also applicable to all types of timber dams. While uplift is not present in any type of timber dam, the submerged weight should

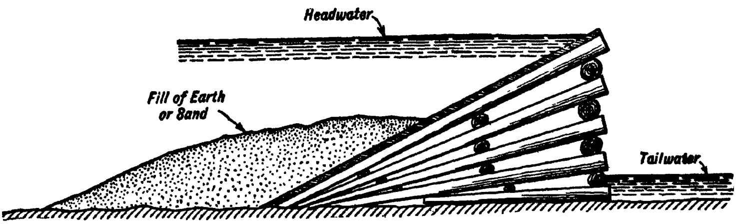

FIG. 116.—Beaver Type of Timber Dam.

be used below the elevation of tail-water. The effective weight of submerged rock fill is:

$$w_s = (w - 62.5)(1 - p), \quad \quad (73)$$

where w is the weight of 1 cu. ft. of solid stone in air and p is the percentage of voids in the fill. It must be remembered that the percentage of voids in the stone fill, in the small spaces between the layers of timber in the crib work, may be very large, depending upon the care with which the fill is placed and the relative size of the timbers and the large stones. The buoyancy of the timber depends, of course, on the kind of timber used. Frequently as much as 25 per cent of the entire volume of a rock-filled crib dam is composed of timber.

On account of the relative lightness of timber dams, the dimensions necessary to prevent sliding are almost always sufficient to prevent overturning. For stability against sliding, the effective weight of a rock-filled crib dam on a rock foundation, including the vertical water pressure on the upstream face, varies from 2.5 times the horizontal pressure of the water, for unimportant, temporary structures on rough foundations, to four times the horizontal pressure, for important dams on smooth foundations.

The stability of A-frame and beaver dams, which have no rockfill to prevent sliding, depends almost entirely upon the strength of the pins which

fasten the dams to rock foundations, unless the foundation is so rough as to permit a horizontal support for the bottom timbers. Friction of wet timber on stone is very small.

The factors of safety to prevent sliding of timber dams on earth foundations follow closely those recommended for masonry dams on earth.

The timbers of the dam should be investigated for strength to transmit the loads to the foundations. In rock-filled dams, much of the load is transmitted through the rock fill, thus relieving the stress on the lower timbers.

94. Tightening the Foundation.—If the dam rests on a rock foundation, the lagging at the upstream toe should be framed as closely as possible to the rock and the junction properly sealed. In some cases the rock is carefully cleaned and a layer of concrete deposited against the toe, as indicated in Fig. 114. In other instances, a fill of impervious earth is deposited against the upstream face of the dam, provided the velocity during floods is not sufficient to disturb it. Low dams on silt-laden streams may have a tight layer of sediment deposited against them during the first freshet.

Timber dams on earth foundations, without adequate sheet piling at the upstream toe, are precarious, even though an impervious fill is placed above the dam. Great care should be exercised to obtain a tight bond between the top of the piling and the lagging of the deck, and a splice plate of steel thoroughly fastened by lag screws is advisable, as a slight movement of the dam is likely to loosen the junction. It is also advisable, where sheet piling is used, to provide a vertical upstream face at least 4 or 5 ft. high and allow the sheet piling to lap this face completely, in order to afford better opportunity for fastening it to the dam. This arrangement is shown in Fig. 117.

95. Protection against Erosion.—Spillway dams must be protected against erosion from the overflow, if the foundations are soft. This is usually accomplished by sloping or stepping the downstream face, as indicated in Fig. 117, and providing an apron to protect the foundations. The apron should be a low rock-filled crib with sufficient rock above the bottom timbers to prevent flotation; or, the apron may be anchored to round piling. A row of short piling and a fill of large rock fragments protect the lower end of the apron from being undermined, as shown in Fig. 117.

96. Choice of Type.—The beaver type of timber dam is the lowest in cost if plenty of timber is available. With more expensive timber, the A-frame type is usually the cheapest. Brush-topped beaver dams are seldom used for permanent structures.

The advantage of the beaver and A-frame types over the rock-filled type is found in their smaller first cost and lower maintenance charges. Rock-filled dams are hard to repair, as the timbers, being buried in the fill, are difficult to replace. The greatest objection to the A-frame type is its danger of failure when neglected. The rock-filled dam is in a large measure supported by the fill and will stand some time after the timbers have become materially decayed.

In the usually remote contingency of nearly complete submergence, which results in negligible head on the crest, the beaver and A-frame dams are likely to float. Neither of these types is easily constructed in deep water, while

crib dams can be partly constructed on land, floated into place, and sunk by filling with stone.

The A-frame type is not particularly suited to earth foundations requiring sheet piling, as the desirability of a vertical upstream face for lapping the sheet piling, as previously described, reduces to a considerable degree the

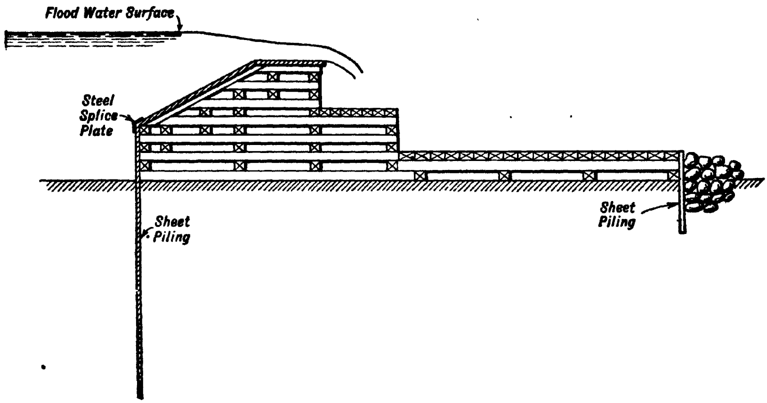

FIG. 117.

length of the sloping deck, the vertical water pressure on which is necessary for the stability of the dam.

97. Limitations of Timber Dams.—Rock-filled timber dams have been constructed successfully to a height of about 70 ft.; but very few dams of this type are higher than 20 ft. The beaver type is limited by the length of the trees available. A-frame dams higher than 20 ft. are seldom encountered.

CHAPTER XII

MASONRY DAMS

By William P. Creager

98. Types.—As available space will not permit of an extensive discussion of the theory of the design of masonry dams, this chapter must be confined to a rather brief synopsis of the subject. The reader is referred to the writer's "Engineering for Masonry Dams"[1] for additional data on the subject.

Masonry dams are divided into the following three general types: (1) solid gravity dams, (2) hollow gravity dams, and (3) arch dams. If the site is not adaptable to an arch dam, the choice between a solid and a hollow dam is chiefly a matter of cost. The hollow dam contains roughly about 35 or 40 per cent of the concrete required for a solid dam, but is considerably more expensive per cubic yard. The hollow dam is frequently found the cheapest for remote localities where timber is available for form lumber and where the transportation cost for cement is great. The reverse is generally true for localities near a railroad where ingredients for concrete aggregate are near at hand. Arch dams are the least expensive of all for locations favorable to that type.

Solid gravity and large arch dams may be composed of ashlar masonry, mortar rubble masonry, or concrete. Large stones are usually embedded in the concrete for these types, and the concrete so constructed is termed "cyclopean concrete." The mix is usually in the proportion of 1 : 3 : 6 throughout, but thinner and richer mixtures have been used.

Thin arch dams are composed of 1 : $2\frac{1}{2}$: $4\frac{1}{2}$ to 1 : $2\frac{1}{2}$: 5 plain or reinforced concrete.

The buttresses of hollow dams are 1 : $2\frac{1}{2}$: 5 to 1 : 3 : 6 reinforced concrete, with decks, aprons, and struts composed of 1 : 2 : 4 reinforced concrete.

99. Nomenclature.—Unless other units are specifically mentioned, all forces are in pounds and all distances in feet.

Let W = vertical force, positive when directed downward;
P = horizontal force, positive when directed toward the left;
P_t = ice pressure per linear foot of dam;
R = resultant of forces;
$\Sigma(W)$ = algebraic summation of the vertical components of all forces acting above a given level, including uplift at that level but not the upward reaction; positive when directed downward;

[1] By W. P. Creager, John Wiley & Sons, publishers.

$\Sigma(P)$ = algebraic summation of the horizontal components of all forces acting above a given level, excluding the reaction at that level, positive when directed toward the left;

$\Sigma(Wx)$ = moment about a given point, of the summation $\Sigma(W)$; positive when counter-clockwise;

$\Sigma(Px)$ = moment, as above, for the summation $\Sigma(P)$;

A = area in square feet;

a = distance from the top of the dam to water surface;

c = ratio of the area subjected to uplift to the whole area;

e = distance from the center of gravity of the base to the point of application of the loading;

$_E$ = subscript used to represent empty reservoir;

$_F$ = subscript used to represent full reservoir;

f = coefficient of static friction as indicated by well-dressed specimens of the materials;

h = vertical distance;

H = total height of a dam above a given level;

I = moment of inertia of a figure in feet units;

k = proportion of voids in earth;

L = top width of a non-overflow dam;

l_0 = known length of a horizontal joint;

l = unknown length of a horizontal joint;

m = distance to the right or left of the center of gravity of a figure;

p = unit pressure or compressive stress in pounds per square foot;

p_r' and p_r'' = unit *vertical reaction*, exclusive of uplift, at the toe and heel of a joint;

p_v' and p_v'' = unit *vertical compressive stress* at the toe and heel of a joint;

p_i' and p_i'' = unit *maximum inclined compressive stress* at the toe and heel of a joint;

p_u' and p_u'' = unit *effective uplift* at the toe and heel of a joint;

p_n' and p_n'' = unit normal pressure of water and earth at the toe and heel of a joint;

q = discharge in cubic feet per second per foot length of crest;

q' = unit load in pounds per square foot;

r = upstream radius of arch dams;

s = factor of safety;

t = thickness of arch dams;

u = horizontal distance from the downstream extremity of a joint to the point of intersection of the resultant R;

w = unit weight;

w_1 = unit weight of masonry;

w_2 = unit weight of water;

w_3 = unit weight of earth;

x = lever arm of moments;

y = horizontal distance from the origin of moments to the upstream extremity of the joint;

z = horizontal distance from the origin of moments to the point of intersection of the resultant R with the joint;

α = angle of repose of earth;

θ = angle of inclination of the resultant R with the vertical; (If the base of the dam is inclined, θ is measured from a normal to the base.)

ϕ' and ϕ'' = angle of inclination, with the vertical, of the face of the dam at the toe and heel.

100. Forces Acting on Dams.—A consideration of the following forces is necessary in the complete design of masonry dams:

(*a*) Water pressure, including uplift;
(*b*) Earth or silt pressure;
(*c*) Ice pressure;
(*d*) Weight of the dam;
(*e*) Reaction of the foundation.

Some of these forces do not admit of exact determination, and certain assumptions must be made for purposes of design. These assumptions must be based on the exercise of the engineer's best judgment and experience, and confirmed by what precedent has shown to be safe.

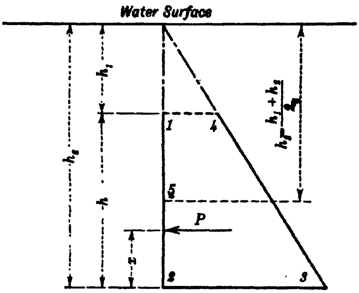

FIG. 118.

Water Pressure.—In Fig. 118, let 1–2 represent a submerged vertical rectangular plane, of area A, and width b. The total pressure, P, of quiet water on each side of this plane is:

$$P = w_2 A h_3, \quad . \quad . \quad . \quad (74)$$

where w_2 is the weight of 1 cu. ft. of water (usually assumed as 62.5 lb. per cu. ft.) and h_3 is the vertical distance from the center of the plane to water surface (all dimensions in feet). This may be reduced to:

$$P = \frac{bw_2}{2}(h_2^2 - h_1^2). \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (75)$$

The force P will be horizontal and will be located a vertical distance above the bottom of the plane equal to:

$$x = \frac{3h_1h + h^2}{6h_1 + 3h}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (76)$$

If h_1 = zero, Eq. (75) reduces to:

$$P = \frac{bw_2h_2^2}{2}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (77)$$

And Eq. (76) reduces to:

$$x = \frac{h}{3}. \quad \ldots \ldots \ldots \ldots \quad (78)$$

The impact of the approaching water against the upstream face of the dam is approximately:

$$P' = \frac{bw_2v^2h}{g}, \quad \ldots \ldots \ldots \ldots \quad (79)$$

where v is the average velocity in the middle of the channel of approach in feet per second, g is the acceleration of gravity = 32.2, and w_2 is the weight of 1 cu. ft. of water. Impact need be considered only for low dams having large discharges. P' may be assumed to act at a distance of $\frac{h}{2}$ above the base.

In the design of dams, it is found convenient to deal with horizontal and

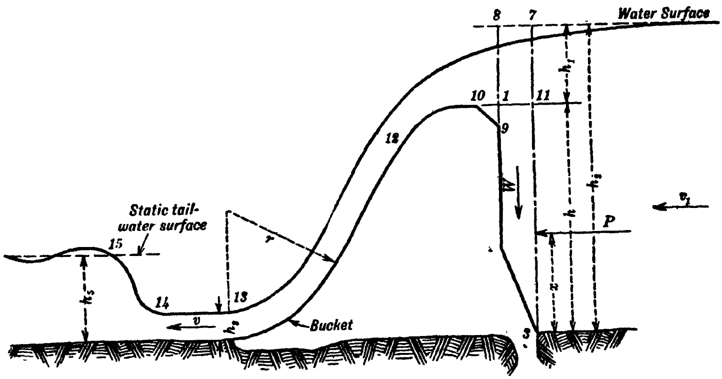

FIG. 119.

vertical forces only. The horizontal component of water pressure on an inclined plane is equal to the water pressure on the vertical projection of the plane.

The vertical component of water pressure on an inclined plane is equal to the weight of water directly above the plane.

Thus, in Fig. 119, the horizontal water pressure, P, on plane 10–9–2–3 is equal to the pressure on the projected plane 11–3, as indicated by Eq. (75), and its distance x above the foundation is found from Eq. (76). The vertical component of water pressure, W, on plane 2–3 is equal to the weight of water within the limits 8–2–3–7, and this force passes through the center of gravity of the area 8–2–3–7.

The vertical component of the water pressure on plane 10–9, as well as the water pressure on the crest and downstream face of the dam due to the spilling water, is neglected, as the jet approaches very nearly spouting velocity.

The horizontal and vertical components of tail-water pressure are found

in the same manner; but, in spillway dams, the energy of the spilling water may be sufficient to reduce the depth of tail-water by the creation of a "standing wave" as indicated in Fig. 119, thereby eliminating tail-water pressure entirely. This may occur if h_5 is equal to or less than about:

$$h_5 = \sqrt{\frac{q^2}{16.1h_3} + \frac{h_3^2}{4}} - \frac{h_3}{2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (80)$$

where q is the discharge in cubic feet per second per foot of crest, and other notation is as indicated in Fig. 119.[2] The dimension h_3 may be obtained as indicated in Sec. 103, Fig. 126.

The presence of tail-water may increase stability or, under certain conditions, may have the opposite effect. Equation (80) is only approximately correct, owing principally to the necessary uncertainty regarding the depth h_3. Therefore, if the depth of tail-water is within 20 per cent of the value given

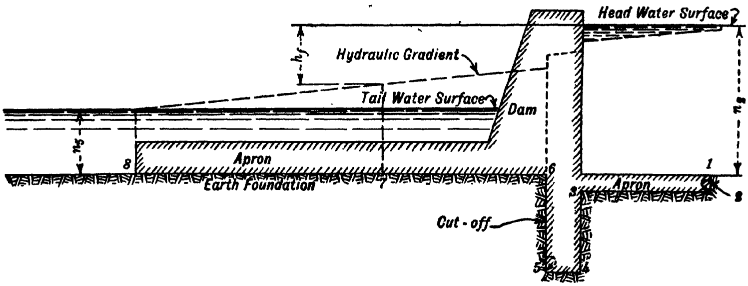

FIG. 120.

in Eq. (80), its resultant pressure on the dam should be considered only if it tends to reduce stability.

In pervious foundations, an upward water pressure, or "uplift," will occur on the base of the dam. In order to understand better the characteristics of uplift, consider the general case shown in Fig. 120 which indicates a dam on an earth foundation, having a cut-off and both an upstream and a downstream apron. It has been proved by experiments that the flow of water under dams has pressure characteristics exactly similar to the flow of water in pipes. The pressure at point 1 is equal to head-water pressure h_2, and that at point 8 is equal to tail-water pressure h_5. The difference, $h_2 - h_5$, is the head lost in friction. The friction loss varies directly as the length of the path of percolation between the masonry and the foundation, as indicated by the hydraulic gradient in Fig. 120. The friction loss between point 1 and any point on the base is therefore proportional to the distance the water has traveled. Thus, at point 7 the friction loss is:

$$h_f = (h_2 - h_5)\frac{l_7}{l_8}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (81)$$

[2] A discussion of the hydraulic jump is given in Sec. 77, and Eq. (80) is identical with Eq. (63), using different nomenclature.

where l_7 is the length of the path of percolation 1–2–3–4–5–6–7 and l_8 the total path of percolation or corresponding distance [3] from point 1 to point 8.

The uplift pressure at any point on the base is therefore equal to the vertical distance from that point to the elevation of head-water surface, less the friction head h_f.

For further discussion of the required path of percolation, including probable leakage in porous foundations and piping, see Sec. 114.

In the case of rock foundations, it is assumed that ony a percentage, c, of the base is subject to uplift pressure. If the foundation has no cut-off, as in Fig. 119, the resultant uplift on the base may be expressed by the equation:

$$W = cw_2bl\frac{h_2 + h_5}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (82)$$

The resultant pressure will be located a distance from the heel equal to:

$$x = \frac{l(h_2 + 2h_5)}{3(h_2 + h_5)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (83)$$

Absolutely impervious foundations are impossible of attainment. The amount of uplift depends upon the relation between the resistance to flow at the upstream end of the base and that over the rest of the base. For this reason a special effort is often made to obtain greater resistance at the upstream end, or heel of the base by one of the following methods:

(1) Increasing the resistance at the heel.

(2) Decreasing the resistance below the heel.

By the first method, trenches are often excavated at the heel and refilled with masonry, or holes may be drilled in the rock and grouted to provide an efficient cut-off. For earth foundations, sheet-piling cut-offs are common, and sometimes upstream aprons are used as indicated in Fig. 120.

By the second method, drainage systems have been installed between the dam and the foundation, downstream from the heel, to allow free exit of the water that passes the heel.

Percentages of c for rock foundations, as high as 0.66, have been used for important structures; but ordinarily a much lower value is adopted. For earth foundations c is always unity.

Uplift should also be considered to exist on all horizontal joints in the masonry above the base. For dams on rock foundations this is usually assumed to be the same as the unit value adopted for the base.

Uplift due to head-water is assumed not to exist in hollow dams, as the cavities in the structure effectively relieve the buttresses of all such pressure.

For a complete discussion of the subject of uplift, see *Transactions* of the American Society of Civil Engineers, Vol. LXXV, pp. 142–225.

[3] Recently there has been considerable discussion regarding the actual length of the path of percolation, some engineers contending that the length of the path is the shortest distance through the foundation between the points 1 and 8, namely 1–2–4–5–8. This is probably correct for very porous foundations; while for very impervious foundations, where the junction of the foundation with the concrete offers relatively much less resistance to flow than does the foundation, the length of path given in the text is more nearly correct.

If it is expected that fine silt will be deposited by the stream against the upstream face and silt pressure included in the forces acting on the dam, it is reasonable to assume that uplift from head-water will not exist. The stability of the dam should be tested with and without the silt pressure.

Earth Pressure.—In practically all streams, considerable quantities of sand, gravel, or silt, washed down by floods, are deposited against the upstream faces of dams, unless special sluicing provisions are made to prevent their accumulation.

The horizontal component of earth pressure may be taken from Rankine's well-known equation:

$$P = \frac{w_3h^2}{2}\left(\frac{1 - \sin \alpha}{1 + \sin \alpha}\right), \quad \ldots \ldots \ldots \quad (84)$$

where P = total pressure in pounds, in addition to the water pressure;
w_3 = weight, in pounds per cubic foot, of the submerged earth:
α = its angle of repose;
h = depth of the earth, in feet.

P is located a distance from the surface of the earth equal to $\frac{2h}{3}$. The unit weight of submerged earth is:

$$w_3 = w_3' - w_2(1 - k), \quad \ldots \ldots \ldots \quad (85)$$

where w_3' = its weight in air;
w_2 = unit weight of water;
k = proportion of voids in the earth.

The nature of the materials deposited against dams varies considerably and should be investigated for each case. Usual values for w_3 are from 60 to 70 lb. per cubic foot; but α varies from about 30 degrees for sand and gravel to zero for liquid mud.

The vertical component of earth pressure on an inclined face is equal to the weight of earth directly above that face, as indicated for water pressure.

Ice Pressure.—Expansion of ice on the surface of the reservoir, due to a change in temperature, may exert an overturning force on the dam. The thrust of ice is impossible of exact determination. An extended discussion of ice pressure is given in the *Transactions* of the American Society of Civil Engineers, Vol. LXXV, p. 142.

Important municipal dams in this country have been designed to resist ice pressures varying from 24,000 to 47,000 lb. per linear foot of dam; but much lower values may be used, particularly if the dam is in a narrow gorge.

Little or no ice pressure usually exists at times of flood. It is therefore seldom necessary to design for ice pressure when water surface is above the level of the spillway crest.

The Weight of the Dam.—The weight of masonry varies considerably, depending upon the ingredients of which it is composed. A common value for use in design is 145 lb. per cubic foot. In important structures, careful

determination should be made of the weight of the masonry to be used, as the amount of materials involved for solid dams varies almost directly as the unit weight.

The Reaction of the Foundation.—In Fig. 121, let $\Sigma(W)$ represent the resultant of all vertical forces acting on the dam, including uplift; and let $\Sigma(P)$ represent the resultant of all horizontal forces. The resultant, R, of $\Sigma(W)$ and $\Sigma(P)$ must be balanced by an equal and opposite reaction of the foundation, consisting of a vertical reaction $\Sigma(W)$ and a horizontal shearing or frictional resistance, $\Sigma(P)$. In the following equations all forces are in pounds, and all dimensions in feet.

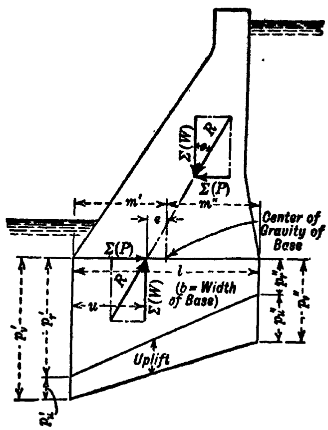

FIG. 121.

The variation of unit vertical reaction is assumed to be linear as indicated in the figure. The unit vertical reaction at the heel and toe is:

$$p_r' = \frac{2\Sigma(W)}{bl}\left(2 - \frac{3u}{l}\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (86)$$

$$p_r'' = \frac{2\Sigma(W)}{bl}\left(\frac{3u}{l} - 1\right) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (87)$$

If the base is divided into three equal parts, the middle part is termed the "middle third." When the resultant, R, intersects the base at the upstream extremity of the middle third:

$$u = \frac{2l}{3}, \; p_r' = 0 \quad \text{and} \quad p_r'' = \frac{2\Sigma(W)}{bl} \quad . \quad . \quad . \quad . \quad . \quad . \quad (88)$$

When the resultant intersects the base at the downstream extremity of the middle third:

$$u = \frac{l}{3}, \quad p_r'' = 0 \quad \text{and} \quad p_r' = \frac{2\Sigma(W)}{bl} \quad . \quad . \quad . \quad . \quad . \quad . \quad (89)$$

When the resultant intersects the base (or any horizontal joint) outside of the middle third, tension will exist at the opposite end. Because of the fact that tension in masonry is unreliable and objectionable, it is the usual practice, in the design of dams, to provide for the resultant intersecting within the middle third. The exception to this rule is in the case of spillway dams, where it is impossible to provide sufficient masonry at the top to force the resultant within the middle third of upper horizontal joints. In such cases, tension is allowed if it is not great; although in some cases steel reinforcement has been provided. Figure 124 indicates the location of the resultant near

the top of a typical spillway dam. In this case reinforcement would not be necessary.

The foregoing equations indicate the unit vertical reaction; but the unit vertical compressive stress exceeds the values given, by the amount of the uplift pressure.

For the resultant within the middle third:

$$p_v' = \frac{2\Sigma(W)}{bl}\left(2 - \frac{3u}{l}\right) + p_u', \qquad (86a)$$

$$p_v'' = \frac{2\Sigma(W)}{bl}\left(\frac{3u}{l} - 1\right) + p_u''. \qquad (87a)$$

For the resultant at the upstream extremity of the middle third:

$$p_v' = 0 + p_u'. \qquad (88a)$$

$$p_v'' = \frac{2\Sigma(W)}{bl} + p_u''. \qquad (88b)$$

For the resultant at the downstream extremity of the middle third:

$$p_v' = \frac{2\Sigma(W)}{bl} + p_u', \qquad (89a)$$

$$p_v'' = 0 + p_u''. \qquad (89b)$$

The foregoing equations are also applicable to the stresses in horizontal joints above the foundations; but to rectangular bases and joints only.

For bases and joints that are not rectangular, the following equations apply:

$$p_v' = + \frac{\Sigma(W)em'}{I} + \frac{\Sigma(W)}{A} + p_u', \qquad (90)$$

$$p_v'' = - \frac{\Sigma(W)em''}{I} + \frac{\Sigma(W)}{A} + p_u'', \qquad (91)$$

where m' and m'' = distance in feet from center of gravity to the extremities of the base;

I = moment of inertia of the base or joint, in feet units, about an axis through its center of gravity perpendicular to the dam section;

e = eccentricity of the loading in feet;

A = area of the base or joint in square feet.

If no tension is to exist. e must have the following value:

$$e = \frac{I}{Am''}. \qquad (92)$$

The unit *vertical* compressive stresses, p_v' and p_v'', are not the maximum compressive stresses, but only the vertical components of such stresses.

Considerable disagreement exists among engineers regarding the maximum

inclined stresses. They do not permit of exact determination; but approximate values may be obtained from the following equations, each indicating stresses in several different planes, the greatest of which is to be used.

At the toe of the dam:

$$p_i' = (p_v' \sec^2 \phi' - p_n' \tan^2 \phi') \text{ or } p_n' \text{ or } p_v' \sec^2 \theta. \quad . \quad . \quad (93)$$

At the heel of the dam:

$$p_i'' = (p_v'' \sec^2 \phi'' - p_n'' \tan^2 \phi'') \text{ or } p_n'' \text{ or } p_v'' \sec^2 \theta. \quad . \quad (94)$$

In the foundation, at the toe:

$$p_i' = p_v' \sec^2 \theta. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (95)$$

In the foundation, at the heel:

$$p_i'' = p_v'' \sec^2 \theta. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (96)$$

The distribution of the force $\Sigma(P)$ over the base cannot be determined. The resistance of the dam to sliding is therefore made to depend solely upon friction which, as will be shown hereinafter, is represented by the inclination, θ, of the resultant with the vertical.

101. Rules of Design.—*Causes of Failure.*—There are only two direct ways in which a gravity masonry dam will fail as a whole, that is, by sliding or by overturning. It may fail by sliding or overturning on a plane above the base, at the base, or on a plane below the base. The last case is likely to occur when the erosive force of spilling water scours out the rock downstream from the dam.

The cause of sliding is the existence of horizontal forces greater than the combined frictional and shearing resistance at the plane of failure.

Overturning occurs when the resultant R passes outside the limits of the dam and the masonry is not capable of resisting tensile stresses. With the resultant well inside the dam, a failure of the masonry by crushing may result in a narrowing of the limits of the dam by an amount sufficient to cause overturning.

Rule 1. *Location of the Resultant.*—The first requirement in a gravity dam is that the resultant shall fall within the limits of the section. It is further required that the resultant shall be so located as to prevent tension in any joint of the dam under all conditions of loading. Eqs. (88) and (89) indicate that tension in rectangular joints is impossible if the resultant intersects within the middle third. For irregular joints the maximum distance of the resultant from the center of gravity of the joint is given in Eq. (92).

Rule 2. *Inclination of the Resultant.*—In order to resist sliding, it is customary to provide that the frictional resistance alone shall be sufficient to resist the resultant of all horizontal forces with ample margin of safety. The frictional resistance is equal to $f\Sigma(W)$, and this should exceed the total horizontal forces $\Sigma(P)$. Expressed algebraically:

$$f\Sigma(W) = s\Sigma(P), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (97)$$

where s is the factor of safety desired.

Therefore:

$$\frac{\Sigma(P)}{\Sigma(W)} = \tan\theta = \frac{f}{s}, \qquad (98)$$

where $\tan\theta$ is the inclination of the resultant R with the vertical.

For rock foundations and horizontal joints in the masonry, s equal to unity may be used, provided that f is taken from the results of experiments on well-dressed specimens of like materials. Roughening of the surface of the foundation and building joints during construction, and the considerable, though indeterminate, shearing resistance are considered to provide the requisite factor of safety.

For masonry on masonry and for masonry on rock, values of f have been assumed variously, between 0.6 and 0.75. For best rock and good workmanship, 0.75 is not excessive; but due allowance must be made for bad conditions and particularly for possible clay seams in the foundations, if the rock downstream is likely to scour away and eliminate the toe hold of the dam.

For gravel, sand, and clay, approximate values of f are 0.5, 0.4, and 0.3, respectively. A factor of safety, s, of 2.5 or more should be adopted for earth foundations unless the dam is anchored by deep cut-off walls or piles.

Rule 3. *Compressive Stresses.*—The compressive stresses in the dam and in the foundation should not exceed allowed limits. Because of a disagreement among engineers regarding the amount of inclined compressive stresses, it has been the practice in the past to prescribe certain allowable vertical compressive stresses and to design for these stresses only. Such vertical stresses, however, have been chosen low enough to compensate for the fact that they do not represent the maximum stresses which exist.

Equations (86) to (96) inclusive, indicate the vertical and inclined stresses in the dam and the foundation. The angle ϕ will not exceed 45 degrees for well-designed high dams, and θ will be considerably less. Consequently it is seen, from Eqs. (93) to (96), that the inclined stresses will not be greater than about twice the vertical stresses. Greater vertical stresses have been allowed at the heel than at the toe of solid dams, because, as ϕ'' is much less than ϕ' and as θ is zero for pond empty, the condition of loading giving the greatest stress at the heel, the inclined stresses at the heel will be much less than at the toe.

The following values of working vertical compressive stresses for masonry dams on good rock foundations will ordinarily result in maximum inclined stresses within safe limits:

At the toe of solid dams, $\frac{1}{21}$ of the ultimate strength;
At the heel of solid dams, $\frac{1}{15}$ of the ultimate strength;
At the toe and heel of hollow dams, $\frac{1}{15}$ of the ultimate strength.

For earth foundations, the requirements for Rule 2 necessitate a very small value of θ. Therefore, the vertical and inclined stresses in the foundation are approximately equal, as indicated by Eqs. (95) and (96), and the stresses in earth foundations govern the design. Common values for allowed stresses in earth foundations are:

Clay	8,000 lb. per sq. ft.
Coarse sand	4,000 to 8,000 lb. per sq. ft.
Fine silt	2,000 to 4,000 lb. per sq. ft.

Rule 4. *Tension in Vertical Planes.*—The inclination, ϕ', of the downstream force with the vertical must be limited to prevent failure by tensile stresses in vertical planes. It is obvious that, if ϕ' is very large, the toe of the dam will be long and tapering and as such will not be capable of transferring the proper proportion of the total load to the foundation without tending to crack off. The writer's empirical equations governing the maximum allowed values of ϕ' are as follows:

For earth or pile foundations:

$$\tan \phi' = < \sqrt{\frac{10}{H}}, \qquad (99)$$

where H is the height of the dam.

For rock foundations:

$$\tan \phi' = < \frac{4f}{3}, \qquad (99a)$$

or

$$\tan \phi' = < \sqrt{\frac{10}{H}}, \qquad (99b)$$

whichever allows the greater value.

102. General Equations for Design of Solid Gravity Dams.—Gravity dams are designed joint by joint, beginning at the top, each joint being made to conform to the foregoing rules of design. Assuming the dam to have been designed from the top to the horizontal joint 1–2, Fig. 122, equations may be written to determine the length and position of the joint 3–4 next below. The designing joints are generally adopted a distance apart equal to about 15 per cent of the distance from the joint to the top of the dam; but a considerably greater distance apart will result in little increased error. The design is thus made progressively from the top to the foundation.

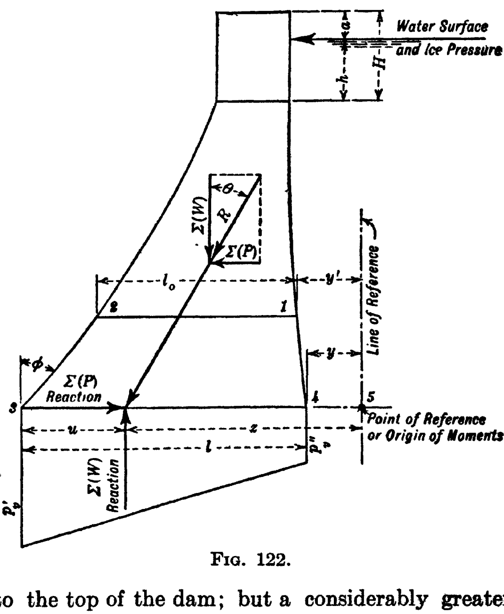

FIG. 122.

In Fig. 123, which represents a typical non-overflow dam, each zone corresponds to a portion of the structure, the design of which is governed by a particular rule or combination of rules.

The top width of non-overflow dams is usually adopted 10 to 14 per cent of the maximum height of the dam, although a greater width may be required for low dams, to withstand shock from floating bodies or to provide a roadway or platform. A superelevation above high-water surface is sometimes adopted for various purposes. A dam with a superelevation of 5 per cent of its maximum height may not contain more concrete than one with no superelevation.

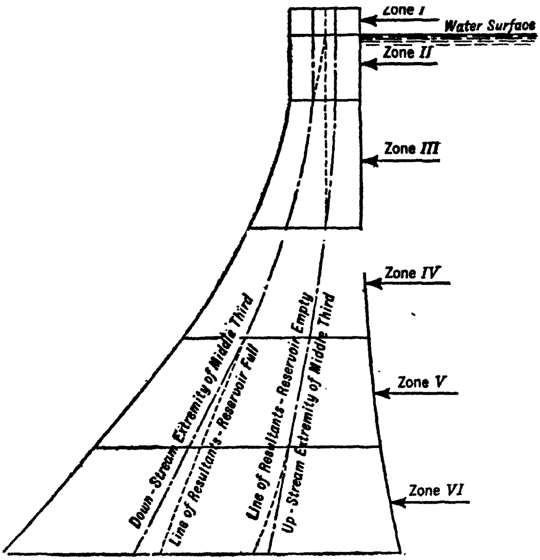

FIG. 123.

Where ice pressure is considered, the weight of masonry in Zone I is fixed by Rule 2 and must be sufficient to prevent that portion from sliding. Either the width of top or the superelevation must be adjusted, if necessary, to this end.

As the top width is always greater than necessary to conform to Rule 1, Zone II represents that portion of the structure in which the resultants, reservoir full and empty, lie within the middle third of the joints, both faces remaining vertical. At the bottom of Zone II, the resultant, reservoir full, is at the downstream extremity of the middle third; while, for reservoir empty, the resultant is at the middle of the joint.

In Zone III the downstream face must begin to batter as indicated in order to accord with Rule 1, the upstream face remaining vertical until the resultant, reservoir empty, intersects the joint at the upstream extremity of the middle third.

In Zone IV both upstream and downstream faces are battered in order that the resultants, reservoir full and empty, may lie at the exact extremity of the middle third.

As the design is continued, the pressures at the toe and heel will increase. The allowed pressures are usually reached first at the toe. In Zone V, therefore, the design is governed by Rule 3 for full reservoir, Rule 1 still influencing the design for empty reservoir. The resultant, reservoir full, will therefore

lie well within the middle third in the lower part of this Zone in order to have sufficient width of joint to distribute the pressures as required by Rule 3.

In Zone VI the allowed working pressures, for both full and empty reservoir, govern, and the design is influenced entirely by Rule 3. In this Zone the resultants for both full and empty reservoir lie well within the middle third. Zone VI will continue until the foundation is reached.

If, at any stage of the design, it is found that $\tan\theta$ exceeds the value allowed by Rule 2, it will be necessary to redesign the section, using an increased batter on the upstream face, in order to include a larger vertical component of water pressure to assist in the resistance to sliding. Ordinarily, for allowed values of $\tan\theta$ not less than 0.7 to 0.75, Rule 2 will not be a governing condition for solid dams except in Zone I as previously indicated.

There is considerable opportunity, in dams several hundred feet in height, for the batter of the downstream face in the lower part of Zone VI to be flatter than that allowed by Rule 4. If this should occur, the remedy lies in an increase in the batter of the upstream face.

In Fig. 124, which represents a typical solid spillway dam, the shape of the crest and downstream face is fixed to conform to the shape of a free-spilling weir jet, as explained later.

In Zone I the requirements of Rules 1 and 2 must of necessity be violated because it is impossible, near the top of a spillway dam, to provide sufficient masonry to withstand sliding by friction alone or to throw the resultant, reservoir full, within the middle third. In Zone I*a* the section still lacks the necessary weight to accord with Rule 2, but the resultants, reservoir full and empty, fall within the middle third in conformity with Rule 1. Unless considerable ice pressure is considered, the height of Zones I and I*a* is small and the tension and horizontal shear can be provided against by monolithic construction above the bottom of Zine I*a*.

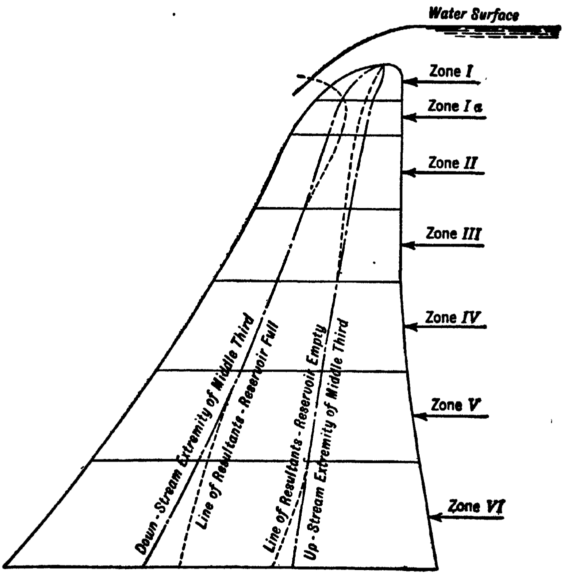

FIG. 124.

When ice pressure occurs, Zones I and I*a* may extend for a considerable distance below the crest. In such cases steel reinforcement may be required

to resist the tensile stresses, and particular care should be taken, with horizontal building joints, to provide numerous large projecting stones or special keyways to increase the shearing resistance.

The conditions affecting Zones II to VI, inclusive, are as previously described for non-overflow dams. In Zone II, however, the downstream face of the spillway sections batters only because of the necessary special shape of the crest and downstream face. The shape of the downstream face, as so fixed, will extend to the bottom of Zone II and its batter will be increased in Zone III.

The arrangement of zones as hereinbefore described may vary somewhat in practice for different types of dams; but the general theory will still apply. In all cases the part of the dam between two horizontal designing joints should be proportioned in accordance with the rules of design that are thought to govern that particular part of the dam, and then tested for stability in accordance with all the other rules.

The basic equations for design, termed "equations of investigation," are employed principally to test any predesigned section for conformity with the rules of design. Further equations, derived from the basic equations and termed "equations of determination," are necessary in determining the length and location of the horizontal designing joints as the design progresses.

EQUATIONS FOR RULE 1.—*Equations of Investigation.*—Referring to Fig. 122, the moment of the resultant, R, of all forces above the joint 3–4, about the point 5, is $\Sigma(Wx) + \Sigma(Px)$. This is equal to the moment of the reaction at the base, or $\Sigma(W)z$, the moment of the force $\Sigma(P)$ being zero. Therefore:

$$\Sigma(Wx) + \Sigma(Px) = \Sigma(W)z,$$

or:

$$z = \frac{\Sigma(Wx) + \Sigma(Px)}{\Sigma(W)} \quad \quad (100)$$

This is the basic equation for Rule 1. It gives the location of the resultant relative to the point 5. According to Rule 1, the distance $z-y$ must be less than $\frac{2l}{3}$ for full reservoir and greater than $\frac{l}{3}$ for empty reservoir.

Equations of Determination.—*Case* 1.—The most general equation is applicable to Zone IV where the location of the resultant for both full and empty reservoir governs the design, and the results for both cases are required to intersect the joint at opposite extremities of the middle third. The two general equations for this case are:

For full reservoir:

$$y + \frac{2l}{3} = \frac{\Sigma(Wx)_F + \Sigma(Px)_F}{\Sigma(W)_F} \quad \quad (101)$$

For empty reservoir:

$$y + \frac{l}{3} = \frac{\Sigma(Wx)_E + \Sigma(Px)_E}{\Sigma(W)_E} \quad \quad (102)$$

Referring to Fig. 122, it is assumed that the dam has been designed down to joint 1–2. Eqs. (101) and (102) will assist in the determination of the length, l, and location, y, of the next joint, 4–3.

In these two equations we have two unknowns, y and l. The values $\Sigma(Wx)$ and $\Sigma(W)$, for both full and empty reservoirs, are, however, dependent upon y and l. The solution of the equations is therefore made by trial. Tentative values of y and l are assumed, from which tentative values of $\Sigma(Wx)$ and $\Sigma(W)$ may be calculated and substituted in Eqs. (101) and (102) to derive more accurate values of y and l. One or two trials will ordinarily be sufficient to determine y and l closely enough for all practical purposes.

Case 2.—For Zone III, in which the upstream face is vertical, y is a constant and l can be determined from Eq. (101) alone, by the method previously described. The length, l, of the joint having been thus determined, the location of the resultant for reservoir empty can be determined from Eq. (100).

Case 3.—The location of the lower extremity of Zone II, for solid non-overflow dams, can be obtained from the following equation, reference being made to Fig. 122:

$$w_2h^3 + (cw_2L^2 + 6P_t - w_1L^2)h = w_1L^2a. \quad . \quad . \quad . \quad . \qquad (103)$$

The location of the bottom of Zones III and IV can be obtained only by trial as the design progresses.

EQUATION FOR RULE 2.—*Equation of Investigation.*—Eq. (98) is the basic equation of investigation for Rule 2.

Equation of Determination.—If, at any stage of the design, it is found that Rule 2 is a governing condition as in Zone I, the required value of $\Sigma(W)$ can be found from Eq. (98), the adopted maximum allowed value of $\tan\theta$ being used.

EQUATIONS FOR RULE 3.—*Equation of Investigation.*—Equations (86*a*) and (87*a*) are the basic equations of investigation for Rule 3.

Equations of Determination.—For Zone V, where Rule 3 at the toe and Rule 1 at the heel govern the design, the following equations of determination apply:

For full reservoir:

$$\frac{(p_u' - p_v')l^2}{6} - \frac{\Sigma(W)_F l}{3} = \Sigma(W)_F y - \{\Sigma(Wx)_F + \Sigma(Px)_F\}. \quad . \qquad (104)$$

For empty reservoir:

$$y + \frac{l}{3} = \frac{\Sigma(Wx)_E + \Sigma(Px)_E}{\Sigma(W)_E}. \quad . \quad . \quad . \quad . \quad . \quad . \qquad (105)$$

Referring to Fig. 122, it is assumed that the dam has been designed down to joint 1–2. Eqs. (104) and (105) will assist in the determination of the length l, and location, y, of the next joint, 4–3. In these two equations we have two unknowns, l and y, but the values $\Sigma(Wx)$ and $\Sigma(W)$, for both full and empty reservoir, are dependent upon l and y. We therefore solve the equations by trial, adopting tentative values of l and y, and solving for closer values as previously described for Eqs. (101) and (102).

For Zone VI, where Rule 3 governs the design at both toe and heel, the following equations apply:

For full reservoir:

$$\frac{(p_u' - p_v')l^2}{6} - \frac{\Sigma(W)_F l}{3} = \Sigma(W)_F y - \{\Sigma(Wx)_F + \Sigma(Px)_F\}. \quad (106)$$

For empty reservoir:

$$\frac{(p_u'' - p_v'')l^2}{6} - \frac{2\Sigma(W)_E l}{3} = \Sigma(W)_E y - \{\Sigma(Wx)_E + \Sigma(Px)_E\}. \quad (107)$$

These equations can also be solved for l and y by the foregoing tentative method.

Equations for Rule 4.—Eqs. (99), (99*a*) and (99*b*) may be used for equations of investigation for Rule 4. No equations of determination can be written. If, at any stage of the design, it is found that the inclination of the face is too small, adjustments must be made in the section, as previously described, in order to steepen the face to the desired angle.

103. Design of Solid Gravity Dams.—The design of non-overflow dams is always started at the top. The superelevation of the top of the dam above high-water surface is generally made sufficient to get beyond the reach of waves and to provide sufficient weight to resist ice thrust. The width of top is usually from 10 to 14 per cent of the maximum height, unless a greater width is required for a roadway, walkway, or other purpose.

The top of the dam having been fixed, the design proceeds according to the foregoing methods.

The first step in the design of a spillway dam is the determination of the elevation of head-water surface above the spillway crest corresponding to the maximum flood to be expected. A discussion of flood probabilities is given in Chapter V, and of the flow over dams in Sec. 74.

After the head on the crest has been fixed, the shape of the upper part of the dam must be determined. If the sheet of water, spilling over the crest of a dam, jumps clear of the downstream face, a tendency to form a vacuum exists under the sheet, unless it has ample access to the atmosphere. A partial vacuum will add to the overturning forces acting on the dam. For this reason, and because it is advisable to avoid impact and scour at the toe, the top of the dam and the downstream face, under usual conditions, should be shaped to correspond to the curve of the underside of the sheet of water due to the maximum flood to be expected. That is to say, the downstream face should have a slope at all elevations no steeper than that curve. For low dams on good rock foundations, this condition may not be required if the depth of head-water over the crest is small and provision is made to prevent a vacuum.

Figure 126 and Table XXXII indicate the recommended shape of the crest and the maximum slope of the downstream face for spillway dams. The dotted lines in Fig. 126 show the upper and lower nappe of the freely flowing water. The full line, indicating the recommended masonry line, is

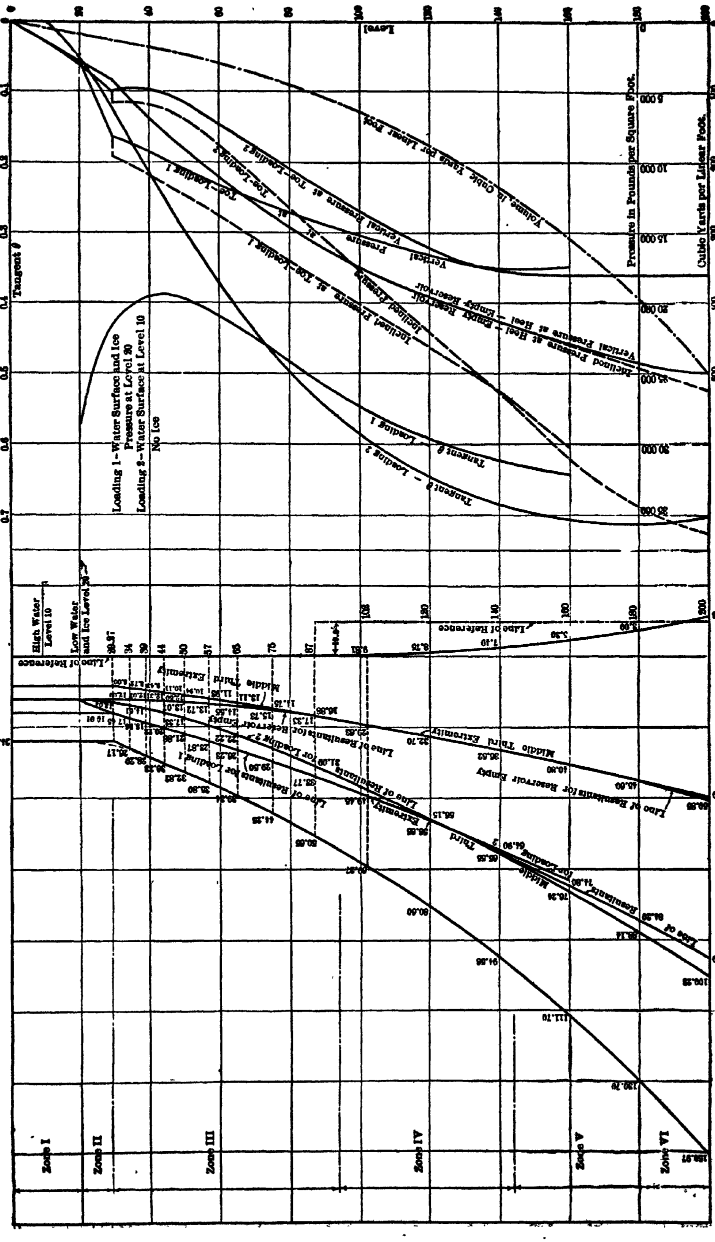

FIG. 125.—Example of Design of Solid Non-overflow Dam.

not as steep as the theoretical water lines on account of the necessity of providing a margin of safety, the water lines being projected, from experiments, relatively close to the crest.

The requirements of Rules 1 and 2 can never be met within Zone I, and stability must be provided by the special construction described in Sec. 102.

The lower limit of Zone II can only be obtained by calculating the location

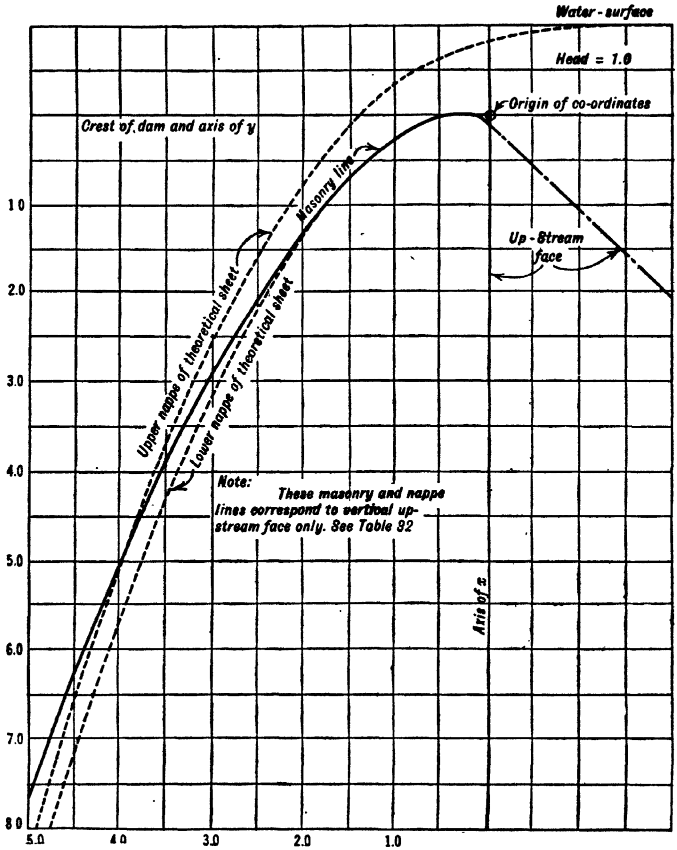

FIG. 126.—Recommended Shape of Crest and Downstream Face for Spillway Dams.

of the resultant, reservoir full, at successive levels until a level is reached where the resultant intersects the joint at the exact extremity of the middle third. The downstream face of the dam will have the shape indicated in Table XXXII until this level is reached, below which the dam, in consideration of other requirements, will have a greater thickness and a flatter downstream face than would be required to fit the sheet of spilling water.

Below the lower limit of Zone II the design is carried out exactly as pre-

viously described for solid non-overflow dams until the level of the foundation is reached. After the shape of the section is determined, a bucket (Fig. 119) should be added at the toe to deflect the spilling water away from the dam. The bucket is sometimes omitted for small dams and for small maximum discharges, if the foundation is hard rock.

TABLE XXXII

VALUES OF COORDINATES IN FIG. 126 FOR UNITY HEAD ON CREST

(Multiply all Quantities in the Table by Actual Head on Crest)

y	x For Vertical Upstream Face			x For Upstream Face Inclined 45°		
	Masonry Line	Theoretical Water Sheet		Masonry Line	Theoretical Water Sheet	
		Upper Nappe	Lower Nappe		Upper Nappe	Lower Nappe
0.0	0.126	−0.831	0.126	0.043	−0.781	0.043
0.1	0.036	−0.803	0.036	0.010	−0.756	0.010
0.2	0.007	−0.772	0.007	0.000	−0.724	0.000
0.3	0.000	−0.740	0.000	0.005	−0.689	0.005
0.4	0.007	−0.702	0.007	0.023	−0.648	0.023
0.6	0.060	−0.620	0.063	0.090	−0.552	0.090
0.8	0.142	−0.511	0.153	0.189	−0.435	0.193
1.0	0.257	−0.380	0.267	0.321	−0.293	0.333
1.2	0.397	−0.219	0.410	0.480	−0.121	0.500
1.4	0.565	−0.030	0.590	0.665	0.075	0.700
1.7	0.870	0.305	0.920	0.992	0.438	1.05
2.0	1.22	0.693	1.31	1.377	0.860	1.47
2.5	1.96	1.50	2.10	2.14	1.71	2.34
3.0	2.82	2.50	3.11	3.06	2.76	3.39
3.5	3.82	3.66	4.26	4.08	4.00	4.61
4.0	4.93	5.00	5.61	5.24	5.42	6.04
4.5	6.22	6.54	7.15	6.58	7.07	7.61

"The masonry line" of Fig. 126, for a vertical upstream face, has been dimensioned in Fig. 127 to facilitate the making of working drawings. Fig. 127 is made for a depth of 1.0 ft. on the crest. The dimensions given in the figure are to be multiplied by the head on the crest.

Figures 130 and 131 give quantities of concrete in solid dams per linear foot, based on frequently adopted assumptions listed thereon.

104. The Design of Hollow Dams.—While hollow dams of many types have been constructed, the usual type consists of a series of parallel, equidistant concrete buttresses covered by a water-tight deck on the upstream side and, for spillways, a downstream apron and bucket to support the sheet of spilling water.

The general theory of design given heretofore for solid dams will apply also to hollow dams. The latter, however, admit of no direct economical methods of design; the shape of the buttresses, type of decking, and other details are worked up in accordance with the judgment of the designer, and the structure tested for conformity with the rules of design herefotore given.

Hollow dams have been constructed on good rock foundations to heights considerably in excess of 100 ft. All hollow dams on earth foundations, should be low and conservatively designed, as unequal settlement is sure to cause stresses for which provision cannot be made in the design.

The choice of top width and superelevation above water surface for hollow

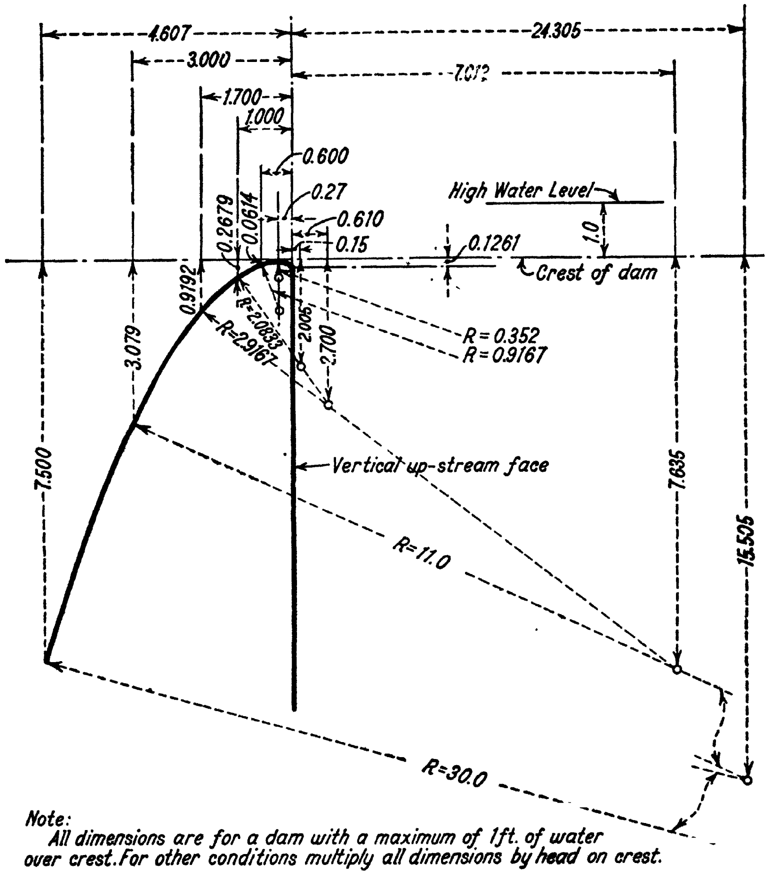

FIG. 127.—"Masonry Line" of Fig. 126 with Dimensions.

non-overflow dams is fixed according to judgment, as outlined for solid dams. The shape of the crest and the upper part of the downstream face of spillway dams must conform, as previously explained, to the shape of the spilling water corresponding to the maximum flood to be expected.

The upstream ends of the buttresses are built on a slope of 45 to 60 degrees with the horizontal, thus allowing the utilization of a large vertical component

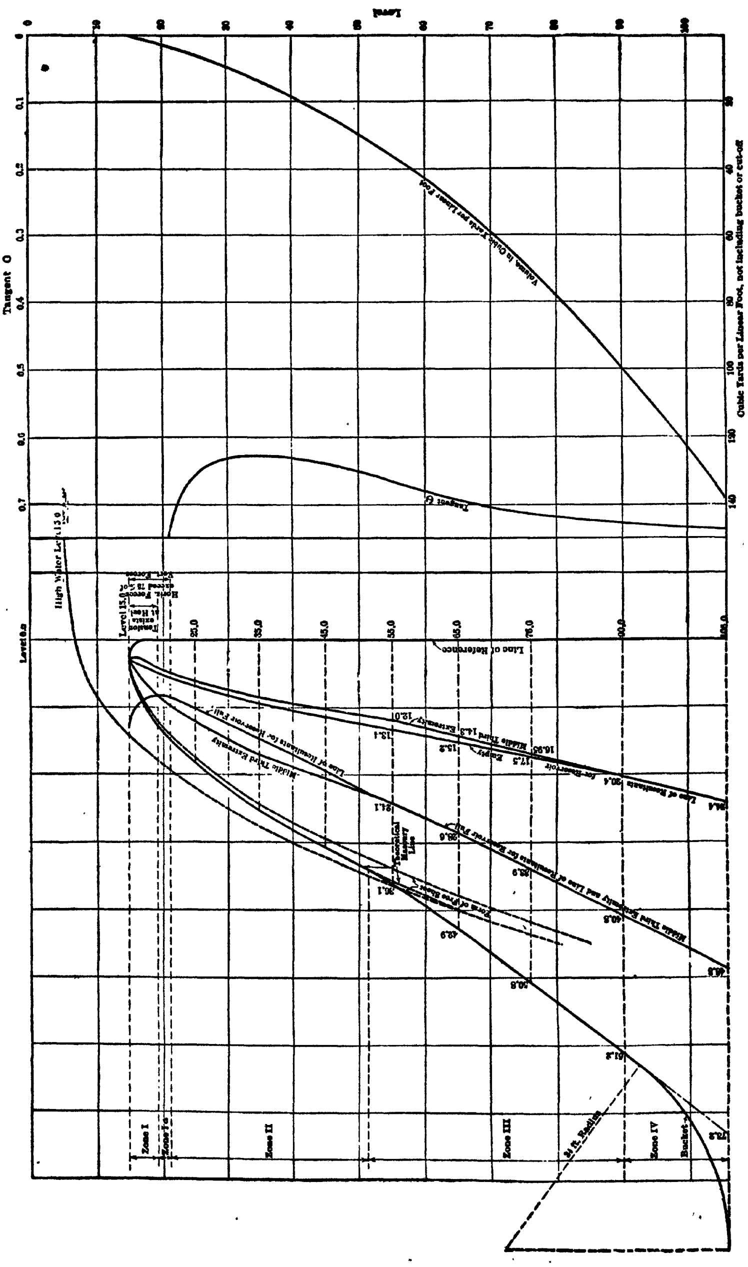

FIG. 128.—Example of Design of Large Solid Spillway Dam.

of the water pressure to assist stability. This feature results in a considerable saving of concrete as compared with that required for a solid dam.

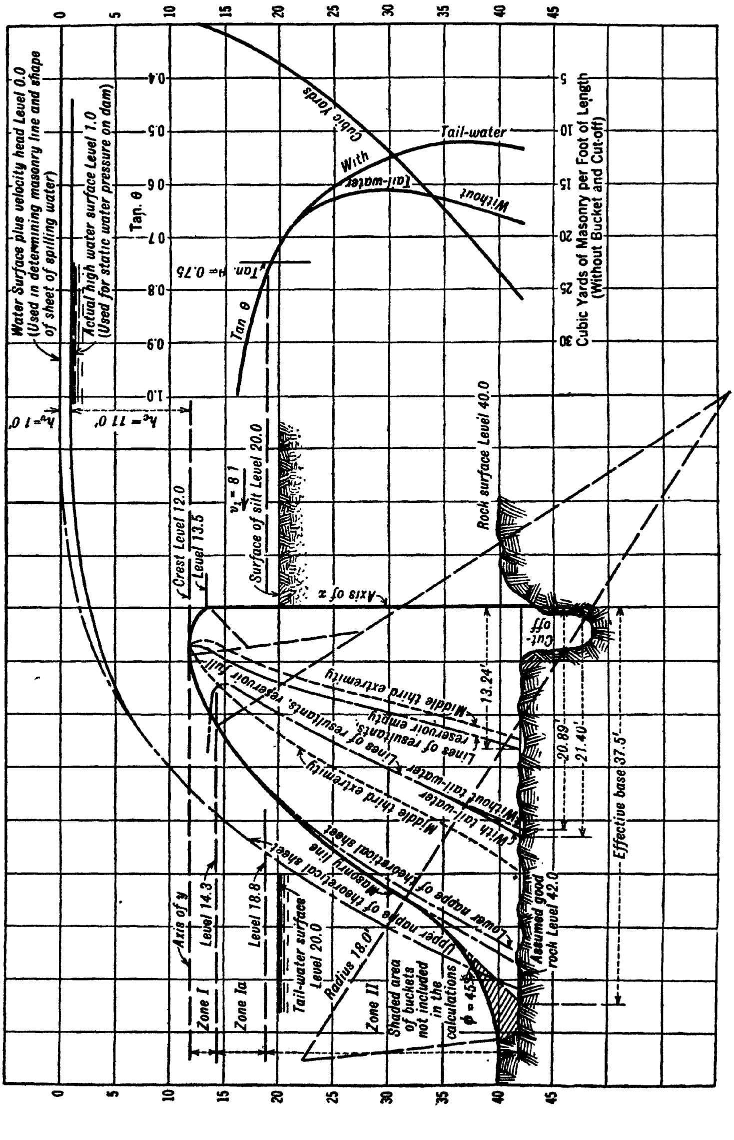

FIG. 129.—Example of Design of Small Solid Spillway Dam.

The downstream ends of those buttresses having no downstream apron are made vertical or slightly battered. This batter usually starts from about

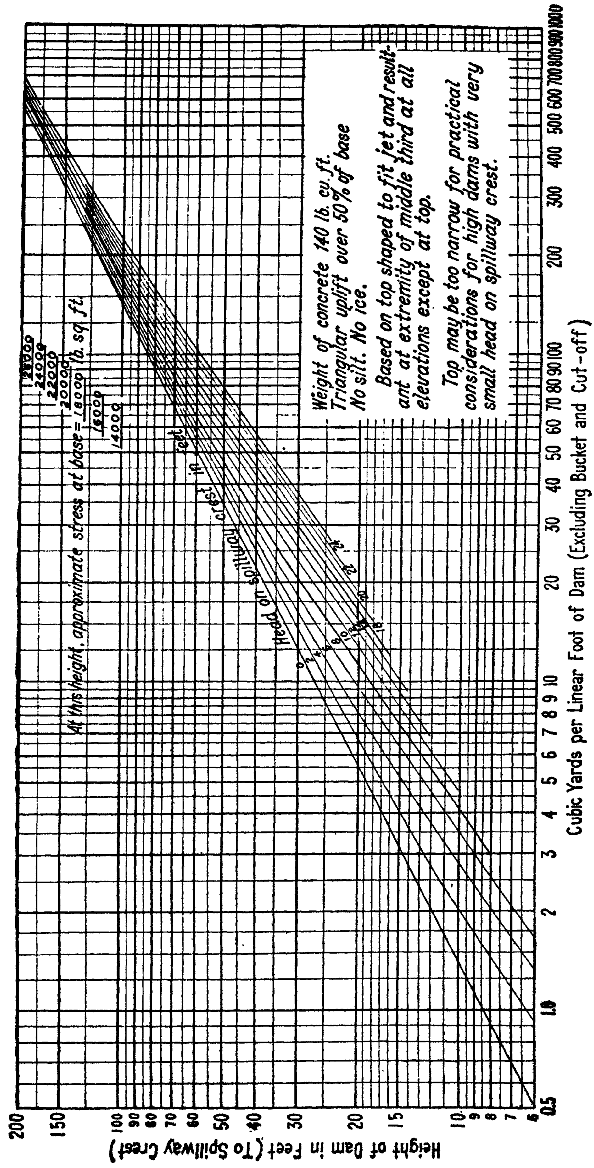

FIG. 130.—Contents of Solid Gravity Spillway Dams.

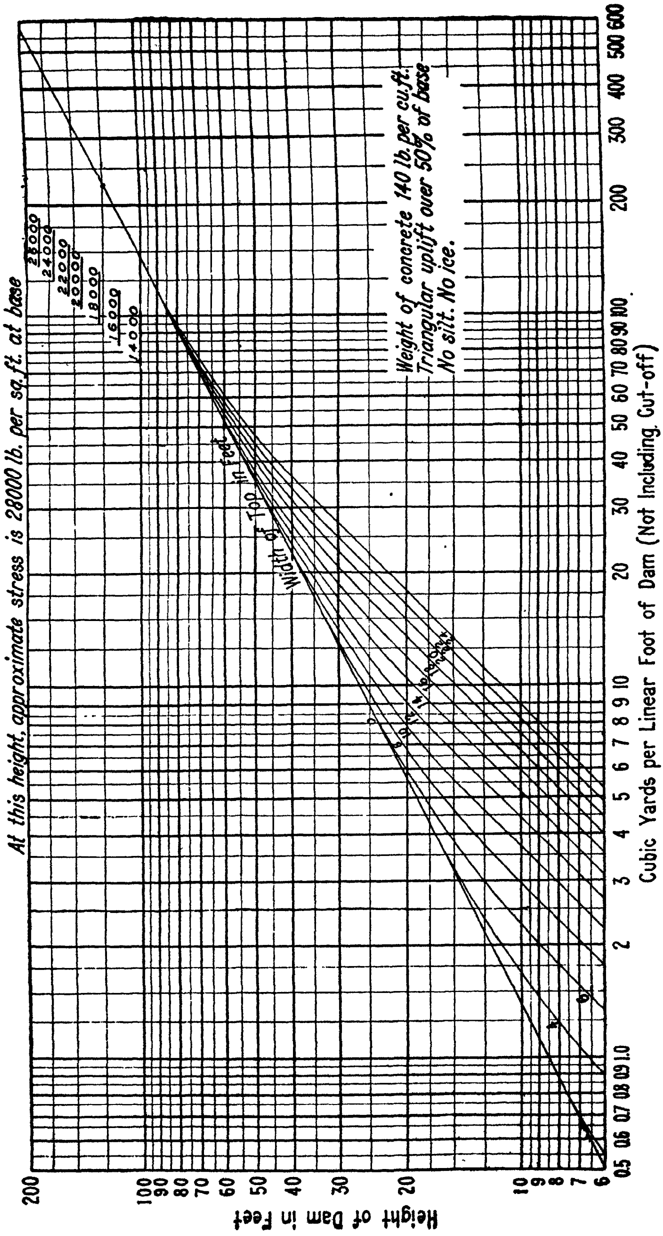

FIG. 131.—Contents of Solid Gravity Non-Overflow Dams.

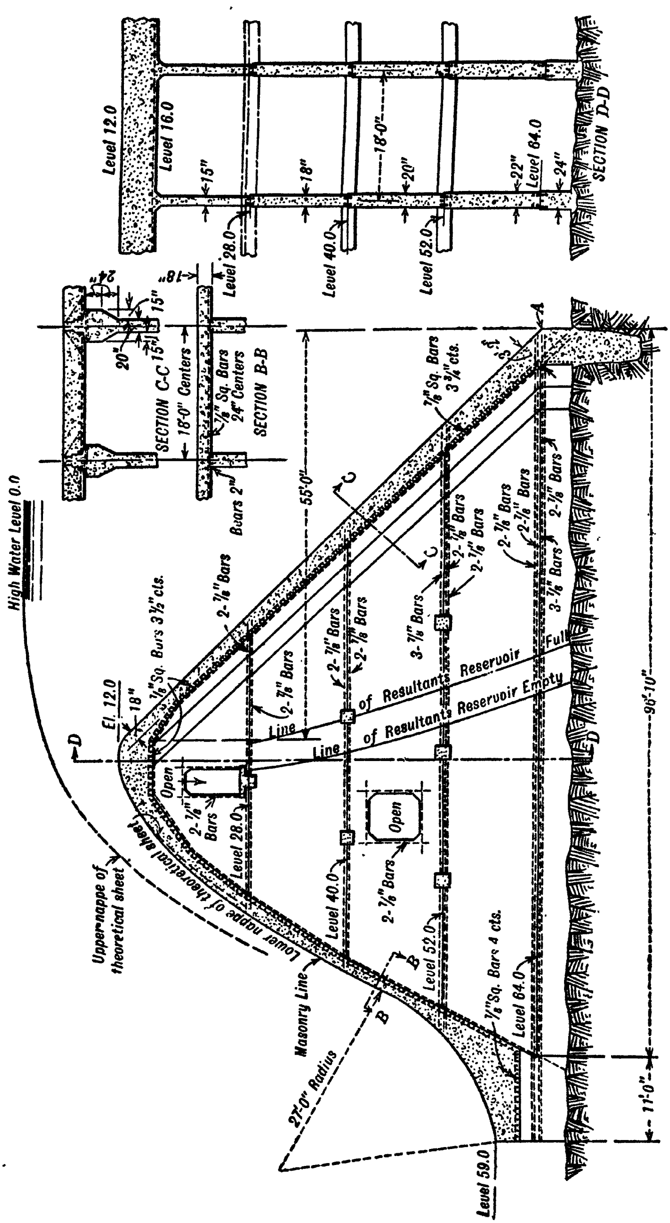

FIG. 132.—Example of Design of Hollow Spillway Dam.

mid-height to three-quarters-height of the dam. A downstream batter may be necessary to conform to the requirements of Rule 1 and, in some cases,

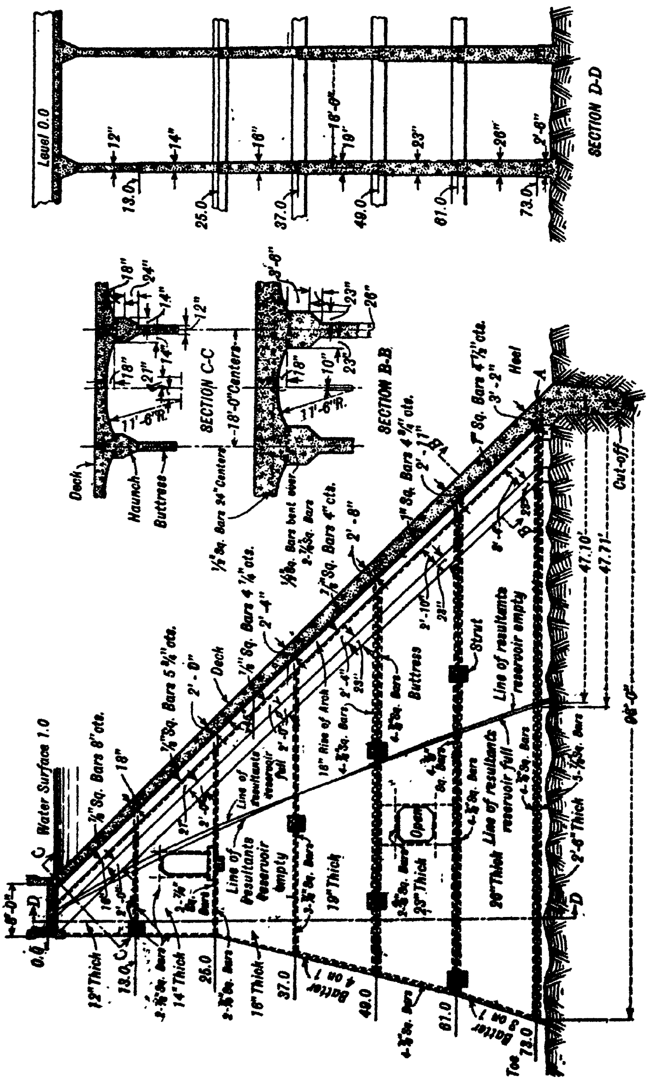

Fig. 133.—Example of Design of Hollow Non-Overflow Dam.

it may be found advantageous to limit the controlling stresses in the concrete by extending rather than thickening the buttresses.

The buttresses are, in reality, thin walls, quite heavily loaded. They are

usually well reinforced for stiffness with steel bars extending into the deck, and provided with struts at intervals, extending throughout the length of the dam. The struts are usually spaced center to center about twelve times the thickness of the buttresses.

The struts are reinforced for compression, for tension, and also for bending due to their own weight and the weight of any interior walkways which they may be required to support. The horizontal building joints in the buttresses are located on the center line of struts.

On soft foundations the buttresses may be provided with plain or reinforced concrete spread footings. These footings are sometimes of sufficient width to provide a continuous concrete mattress under the dam. Such mattresses should be provided with large weep-holes to prevent uplift from headwater.

Open holes in the buttresses are convenient for the passage of men and materials during construction; and an inspection gallery is sometimes provided unless, in low dams, access to the interior may be had from the downstream side at ground level.

There are two general types of hollow dams, the flat-deck type and the multiple-arch type. Each type is characterized by the decking. The deck for the former type consists of flat, reinforced concrete slabs as indicated in Figs. 132 and 133, while the deck for the multiple-arch dam consists of a series of arches as shown in Fig. 134.

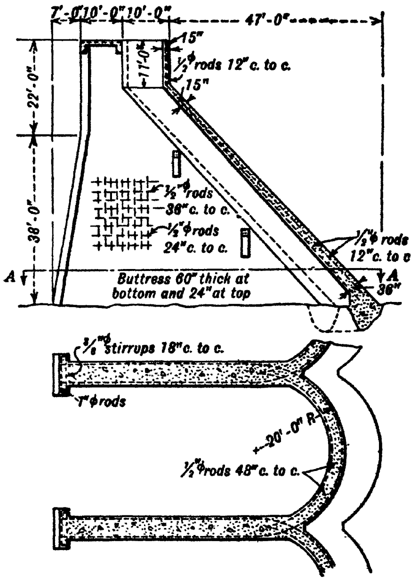

FIG. 134.—Typical Hollow Multiple-arch Dam.

The loading for the flat slab deck is uniform for each elevation. Arch decks, however, are designed in normal slices and, for each slice, the crown is at a higher elevation than the haunch. Consequently, the loading will be less per square foot at the crown than at the haunch. This is an important consideration, particularly near the top of the dam where the maximum percentage difference occurs. Mainly for this reason, the top parts of multiple-arch dams are usually provided with vertical decks, and some recent dams are designed and built with the arches circular in horizontal section.

The component of the weight of the deck, acting normal to the upstream edge of the buttresses, should be included in the deck loading.

The arches of multiple-arch dams should always be reinforced for stiffness and well tied into the buttresses, even if such reinforcement is not needed to

resist the unequal loading of the arch. A central angle of 120 degrees is considered most economical for the arches, as explained hereinafter for arch dams.

With flat decks the buttresses are spaced from 15 to 25 ft. centers; but for arched decks a greater spacing is more economical, 30 to 40 ft. being usually adopted. Much depends upon the average height of the dam and other considerations. The most economical spacing can only be obtained by comparisons of alternative designs.

For hollow spillway dams of considerable height and those on foundations susceptible to erosion, the dam is provided with a downstream apron, shaped to fit the sheet of spilling water corresponding to the maximum flood to be expected. The proper shape is indicated in Table XXXII and Fig. 126. The apron should terminate in a bucket at the toe of the dam, to deflect the water properly in a horizontal direction. The apron, and particularly the bucket, should be of ample thickness and well reinforced to withstand the impact of ice, trees, and other objects which may pass over the crest.

If an apron is provided, provision should be made for thorough ventilation of the interior of the dam by means of openings in the buttresses and an open shaft at each end. The interior of the dam should be well drained to prevent the accumulation of leaks from head-water.

In the design of hollow dams the choice of type is the first consideration. This varies with local conditions and the judgment of the designer.

The spacing, thickness, inclination, and other details of the buttresses and deck are then adopted tentatively, and the dam is tested for conformity with the rules of design hereinbefore given.

Rule 1 may necessitate a change in the length of the buttresses, either by flattening the slope of the downstream end to increase the length of the base, or by flattening the slope of the upstream end to increase the vertical component of the water pressure, whichever seems the more expedient.

Rule 2 may require additional weight, which can best be obtained by providing a flatter batter to the upstream face.

Rule 3 may require an increase in area of buttresses by thickening or lengthening or both. When calculating the unit compressive stresses in horizontal joints and the base, the horizontal area of the deck, not being in monolith with the buttresses, should be included if, when this is done, the calculated stresses are found to be larger. Referring to Sec. 100, it is seen that the addition of the area of the deck to the area of the buttresses will decrease the direct stress, but the eccentricity, e, may be lengthened sufficiently to increase the flexural stress to a greater extent, resulting in an increased total stress at the extremity of the base.

Rule 4 is not a governing condition in the usual type of hollow dams.

Uplift due to head-water is never considered for hollow dams, on account of the thorough draining of the interior and the inclined reinforcement in the decks, which is carried down into the masonry cut-off.

The designing conditions and characteristics of the hollow dams indicated in Figs. 132 and 133 are given in Table XXXIII. The weight of concrete was taken at 150 lb. per cubic foot.

TABLE XXXIII

CHARACTERISTICS OF HOLLOW DAMS INDICATED IN FIGS. 132 AND 133

Levels	SPILLWAY DAM				NON-OVERFLOW DAM					
	64	52	40	28	73	61	40	37	25	13
Deck reaction, lb. sq. ft.	39,000	35,000	30,000	26,000	37,500	35,000	34,000	30,500	23,500	13,000
Max. vertical stress	29,000	27,000	22,000	21,000	31,000	31,000	31,000	29,000	24,000	14,000
Max. inclined stresses	33,000	29,000	31,000	35,000	42,000	42,000	42,000	38,000	30,000	17,000
Tan θ	0.57	0.55	0.53	0.49	0.60	0.60	0.58	0.55	0.48	0.33
Cu. yd. per lin. ft. of dam *	21.5	15.0	9.5	5.0	26.5	19.5	13.0	8.5	5.0	2.5
Lb. steel per lin. ft. of dam *	1,850	1,350	850	500	1,700	1,300	950	650	450	200

* Not including bucket or cut-off. Includes downstream apron to bottom of bucket concrete. Not including steel in bucket.

105. The Design of Arch Dams.—Arch dams are designed to resist the force of the water and silt by horizontal arch action, and are adaptable only to those sites where the length is small in comparison with the height, and the sides of the valley are composed of good rock to resist the arch thrust at the haunches.

The dam will not act as a true arch because of the restraint at the base where it is in contact with the relatively rigid foundation. Because of this restraint the dam acts partly as a vertical cantilever; this reduces the load to be carried by the lower part of the arch, but transfers an additional load to the upper part.

Many attempts have been made to devise methods of design to take into consideration the effects of the elasticity of the vertical cantilever beam action, the weight of the concrete, varying span, varying radius, and expansion and contraction of the masonry due to changes in temperature, moisture contents and the setting of concrete.

So far, however, such methods of design have not met with general approval as it is considered that the effect of most of these conditions is indeterminate.

Most arch dams have therefore been designed in accordance with the following equation, which applies to thin, submerged, empty cylinders:

$$p = \frac{q'r}{t}, \qquad \qquad (108)$$

where, for a given level,

p = stress in the arch in pounds per square foot;
q' = load on the arch in pounds per square foot;
r = upstream radius in feet;
t = thickness of the arch in feet.

It must be realized that this equation is inaccurate for arch dams and gives results which are not necessarily on the safe side. Therefore, the allowed arch stress, p, for use in the design must be extremely conservative. Arch

stresses in thirty-four existing dams, computed by Eq. (108), range from 23,000 to 70,000 lb. per square foot, and in two extreme cases 120,000 lb. per square foot is indicated.

There is no record of a failure of an arch dam. For this reason it is not known how closely the actual stresses in these dams approach the ultimate strength of the masonry. It is recommended that the value p, for use in Eq. (108), should not exceed one-eighth to one-twelfth of the ultimate strength of the masonry, depending upon the importance of the structure.

The designer should make a detailed study of existing arch dams and conform to the usual relative dimensions of such structures or reduce his allowed stresses accordingly, as any deviation from existing types should be considered in the nature of an experiment.[4]

As an arch dam is in reality a long column, receiving lateral support only through its connection to the foundation, a maximum ratio of curved length to thickness should not be exceeded. Using as a precedent a study of a number of existing dams, the author recommends that this ratio should not exceed 25 at mid-height or 65 at the top of the dam, and preferably should be 20 and 50 respectively. The ratio at the top may be somewhat increased if the ratio at mid-height is considerably decreased, particularly if vertical reinforcement is provided.

The rock at the abutments should be well stepped to withstand the arch thrust safely without sliding at the arch haunch.

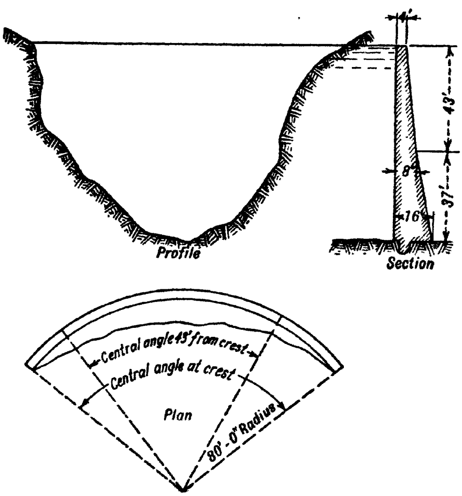

Fig. 135.—Typical Constant Radius Arch Dam.

Horizontal and vertical reinforcement has been used in small, thin arch dams, but seldom to the extent necessary to affect materially the strength of the structure.

Ice thrust, if probable, may be considered as acting over a portion of the arch at least equal to twice the arch thickness plus the assumed thickness of ice, and considerably more if vertical steel reinforcement is used.

Early arch dams were constructed mostly with a constant radius at all levels, necessitating a central angle (Fig. 135) which gradually decreased toward the base of the dam. Jorgensen[5] has shown that the greatest theoretical economy obtains when the

[4] For detailed dimensions of thirty-four arch dams see Table XXIV, "Engineering for Masonry Dams," by W. P. Creager, John Wiley and Sons, 1917.

[5] Trans. Am. Soc. C. E., Vol. 78, p. 685.

central angle is 133° 34′ at all elevations. If the cost of formwork and similar items were included, this angle would probably not exceed 120° or even less.

It is not always possible to use a constant central angle where the configuration of the rock surface is such that a constant angle would result in less

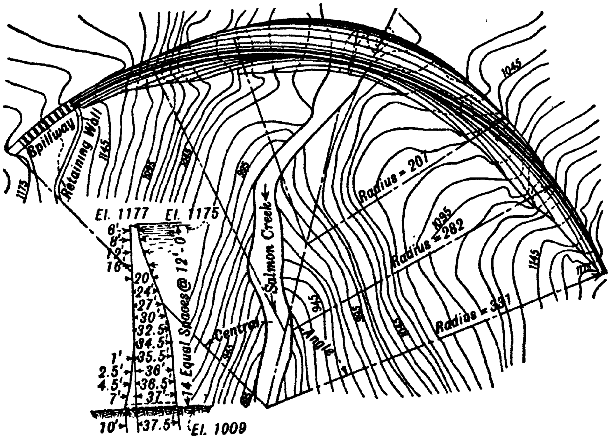

Fig. 136.—Typical Constant Angle Arch Dam, 168-ft. high for Salmon Creek, Alaska.

thickness at the bottom than at higher elevations. Therefore the dam should be designed to as near a constant angle as practical considerations will allow.

For the dam shown in Fig. 136, the central angle reduces materially toward the base, to keep the structure from overhanging in places. However, the dam more nearly conforms to the characteristics of a constant-angle dam than to those of a constant-radius dam. The maximum stress in this dam, calculated from Eq. (108), is 47,500 lb. per square foot.

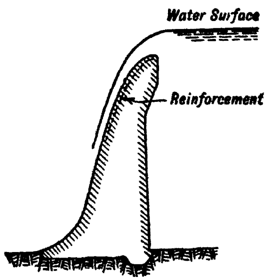

Fig. 137.

The weight of masonry in arch dams is not included in the forces resisting overturning and is therefore ample to resist all possible uplift pressure. In large arch dams, the base is thickened by providing a downstream batter, as indicated in Fig. 136, in order to safeguard against excessive vertical compressive stresses due to vertical beam action.

The downstream face of constant-angle arch dams is usually vertical, or nearly so. A discharge over the crest will therefore leave the downstream face and cannot be directed horizontally away from the dam by a bucket as in the case of gravity dams. Therefore, arched spillway dams of this type are not practic-

able for large discharges unless the foundation is not easily eroded, or proper precautions are taken to protect it.

The safety of an arch dam is dependent mainly upon the strength of the abutments, and particular care should be taken to protect that part from erosion. In some instances, the downstream face has been shaped to fit the underside of the spilling sheet, as indicated in Fig. 137, but this results in a section much too thick for economy as an arch, although some saving can be made by overhanging the lip of the crest.

CHAPTER XIII

EARTH DAMS

BY JOEL D. JUSTIN

106. General.—Lack of space prevents the inclusion in this chapter of all the theory and practice pertaining to the design and construction of earth dams. Therefore the following discussion is confined to a general synopsis of the subject. The reader is referred to Sec. 123, "Bibliography," for further references, and especially to the paper by Joel D. Justin, entitled "The Design of Earth Dams," and published in the *Transactions* of the American Society of Civil Engineers, Vol. LXXXVI. Many of the data on the theory of design given in the present chapter are taken from this paper.

An earth dam was at one time looked upon, by many engineers, as merely a fill of dirt; but scant thought was devoted to its design and but little supervision given to its construction. Avoidable failures, however, riveted attention on it as an engineering structure, and to-day thorough investigations and studies are made of the site of a proposed earth dam and of the materials available for construction. It is also becoming generally recognized that the successful construction of an earth dam requires just as careful attention to details as the construction of other engineering structures.

107. Materials in Earth Dams.—It is quite generally believed by those unfamiliar with the subject that, to be safe, an earth dam must be built of impervious material. Many of the textbooks still state that a dam should not be built unless an impervious material is available. That such is not the case will be evident to anyone who cares to examine the list of successful earth dams. Earth dams have been built successfully of loose rock, of sand of all degrees of fineness and coarseness, of gravel and silt, as well as of rock flour, soil, and clay. Assuming a thorough knowledge of all the available materials, it is possible to design and build a dam of such materials within safety.

108. Line of Saturation and Hydraulic Gradient.—The line of saturation may be defined as the uppermost line of flow of the water through the dam and subsoil. The hydraulic gradient, as applied to earth dams, may be defined as the gradient of a line joining the highest points to which water would rise in a series of pipes placed in the cross-section. This line is generally, but not necessarily, coincident with the line of saturation.

109. Criteria for the Design of Earth Dams.—The practical criteria for the design of earth dams may be stated briefly as follows: An earth dam should be designed so that:

(1) The spillway capacity is so great that there is no danger of overtopping;
(2) The line of saturation is well within the downstream toe;
(3) The upstream and downstream slopes are such that, with the materials used in the construction, they will be stable under all conditions;
(4) There is no opportunity for the free passage of water from the upstream to the downstream face;
(5) Water that passes through and under the dam will have a velocity so small that there will be no undue waste of water and that this water will be incapable of lifting any of the material of the foundation when it rises to the surface below the toe;
(6) The free-board is such that there is no danger of overtopping by wave action.

An earth dam, designed and built to meet these criteria, will prove as permanent as any of the works of man if proper attention is given to important details during construction.

110. Criterion 1.—An earth dam should be designed with the spillway capacity so great that there is no danger of overtopping. One frequent cause of the failure of earth dams is the use of spillways of insufficient capacity. A masonry dam with an insufficient spillway will generally stand overtopping to a considerable depth without serious damage; but with an earth dam, overtopping means practically certain failure. Many earth dams are in use, that have spillways of insufficient capacity to care for floods which are certain to arrive sooner or later. The subject of flood flows is discussed in detail in Chapter V.

111. Criterion 2.—An earth dam should be designed so that the line of saturation is well within the downstream toe.

Factors Affecting the Position of the Line of Saturation.—The position and slope of the line of saturation in the cross-section of an earth dam is affected by many factors, such as:

(1) The porosity of the soil composing the dam and its foundation;
(2) The effective size of the particles composing the material of the dam and its foundation;
(3) The distribution of particles of various effective sizes throughout the cross-section;
(4) The depth of soil below the base of the dam;
(5) The flow of ground water and its depth below the original surface at the site;
(6) The nature and depth of soil downstream from the dam;
(7) The existence of a core-wall, puddle-wall, or apron within the dam itself;
(8) The use of drains for collecting seepage in the downstream part of the dam.

Change in Ground-water Level.—The construction of an earth dam will often have a marked effect in increasing the level of the ground water downstream from the dam. Land that was previously dry may become marshy and springy. This is sometimes the occasion for damage suits. If the ledge rock

is located near the surface at the site, this fact will cause the line of saturation in the earth dam to be higher than it would be if rock were at a greater depth. Ledge rock near the surface sometimes causes the saturation of the downstream toe. Where ledge is at or near the surface, the precaution is often taken of placing a heavy rock fill against the toe.

Effect on Line of Saturation of Making Down-stream Part of Dam More Pervious than Upstream Part.—There is an advantage in selecting and segregating the materials so that the upstream part of the dam will be less pervious than the downstream part. In Fig. 138 the material composing the foundation is assumed to be a fairly tight soil, and the upstream part of the dam to be built of very impervious clay, whereas the downstream part is assumed to be of quite pervious material. The effect of this arrangement is to bring the line of saturation much farther within the downstream toe than would be the case if the structure were composed entirely of such impervious material as that in the upstream part. It will be noted that the line of saturation is much steeper in the pervious material. If the dam were to be built entirely of the same impervious material, the slope of the line of saturation would be uniform. In this arrangement, the function of the upstream part is to keep as much of the water as possible out of the dam, and the function of the downstream part is to drain away as rapidly as possible such water as passes through the upstream part, thus greatly increasing the stability of the structure over what it would have been had impervious material been used throughout the entire cross-section.

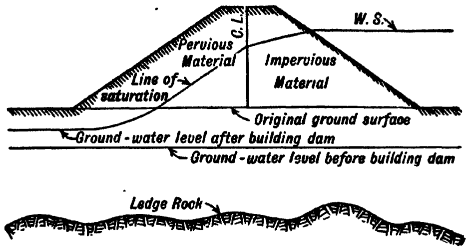

FIG. 138.—Earth Dam with Pervious Downstream Section and Impervious Upstream Section.

Effect of Core-wall in Dam of Fairly Pervious Material on a Foundation of Like Material.—In Fig. 139 the dam is assumed to be built of fairly pervious material which is practically identical with the foundation soil at the site. The core-wall, whether of masonry or puddle, is assumed to be impervious and to extend only a slight distance below the original surface of the ground. Such a core-wall does little good. Although the water does not pass through the wall, it does pass under it; its head will be only slightly decreased by the core-wall, and the line of saturation will be little, if any, different from what it would have

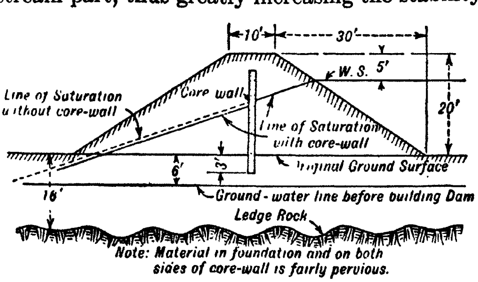

FIG. 139.—Core-wall in Dam of Pervious Material on Foundation of Similar Material.

been without a core-wall. In some instances the drop in the line of saturation at the core-wall is only 2 or 3 ft. In most cases of this kind the money spent on a core-wall is wasted.

Effect of Core-wall in Dam of Pervious Material on a Foundation of Impervious Material.—In Fig. 140 conditions are the same as those described for Fig. 139, except that the foundation material is quite impervious. The core-wall, which is assumed to be impervious, is extended down only far enough to make a good seal with this material. In order to enter the downstream part of the dam, the water must pass under the core-wall and through quite impervious material, which causes a relatively great loss of head. The discharge is consequently much less than that occasioned by the conditions shown in Fig. 139. The line of saturation thus shows a marked drop at the core-wall. In this case the wall is important, as without it, the line of saturation would be about as indicated in Fig. 139 (" Line of saturation without core-wall "). In fact, without the wall, it is conceivable that the line of saturation might fall outside the base and the structure be entirely unsafe, whereas a dam with a wall well set into the impervious foundation material is safe.

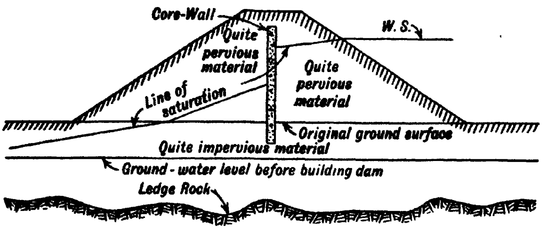

FIG. 140.—Core-wall in Dam of Pervious Material on Foundation of Quite Impervious Material.

Effect of Extending Core-wall into Pervious Foundation.—Fig. 141 is a

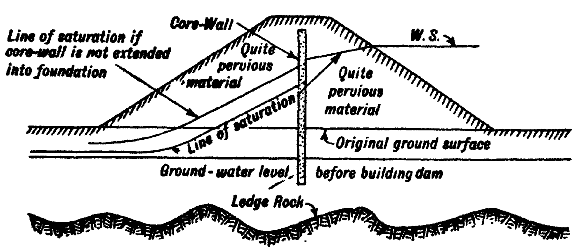

FIG. 141.—Showing the Effect of Extending the Core-wall into a Pervious Foundation.

cross-section of an earth dam at a site where conditions are identical with those shown in Fig. 139, except that the core-wall has been extended much farther into the pervious foundation. The core-wall is assumed to be impervious, and the water, in passing to the downstream side of the core-wall, must traverse a greater distance than in Fig. 139. The frictional resistance is proportionately greater, and the water rises on the downstream side of the core-wall to a lesser height than was the case in Fig. 139. The line of satura-

tion is lowered, and the stability of the structure increased. If the core-wall had been extended into the foundation by the driving of a wall of sheet piling, the effect would have been the same except that such a wall would not be so likely to be tight.

It is often assumed that the water, in passing under a wall, flows down the upstream side and then up the downstream side, thus making the distance traversed by the water twice the length of the sheet piling or wall. Mr. J. B. T. Colman has shown, in his paper on "The Action of Water under Dams,"[1] that this theory is incorrect, and that the loss of head, due to the insertion of sheet piling at the heel of a dam, takes place almost entirely on the upstream side of the sheet piling.

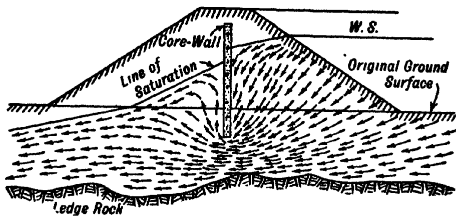

FIG. 142.—Showing the General Condition of Flow through an Earth Dam and under a Core-wall.

What actually happens is about as follows: The water, flowing under pressure through the soil of the foundation, comes in contact with the extended core-wall, which restricts the discharge area through the soil. As the water must flow through this restricted area the velocity and the loss of head accordingly increase. After passing the wall, some of the water rises, and this causes additional loss of head. Most of the water, however, travels in a generally horizontal direction, as illustrated in Fig. 142.

Effect of Extending Core-wall to Ledge Rock.—If the core-wall had been properly bonded to ledge rock the effect on the lowering of the line of satura-

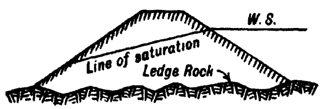

FIG. 143.

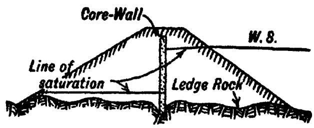

FIG. 144.

FIGS. 143 and 144.—Showing the Effect of Placing a Core-wall in an Earth Dam Founded on Ledge Rock.

tion on the downstream side of the core-wall would have been much more marked, and, theoretically, there would be no water on the downstream side. However, as there are usually some seams in the rock and the concrete or puddled core-wall is never entirely impervious, there is some saturation on the downstream side. Whether or not it is desirable to extend the core-wall to rock can be determined only after a study of all the conditions.

Effect of Core-wall in a Dam Founded on Ledge Rock.—Fig. 143 is a cross-section of an earth dam founded on ledge rock. With a fairly impervious material in the dam, the line of saturation might intersect the downstream

[1] Trans. Am. Soc. C. E., Vol. LXXX (1916), p. 421.

face at a point considerably above the downstream toe. The dam illustrated in Fig. 143 would probably fail. Fig. 144 shows the same dam at the same site with a core-wall added. Note that the line of saturation intersects the downstream face at the ledge. A small quantity of seepage will take place at the toe, under these conditions, but may be cared for by proper drainage. The structure shown in Fig. 143 was made safe by the addition of a core-wall. Some earth dams that have been founded directly on ledge rock have been made safe by an elaborate system of drainage, but, ordinarily, it is not good practice to build such a dam without a core-wall.

Effect on Line of Saturation of Placing Core-wall Upstream from Center of Dam.—The position of the core-wall in the cross-section of the dam is of great importance. It is frequently placed upstream from, instead of directly on, the center line, as in Fig. 141. The conditions for the dam shown on Fig. 145 are identical with those for Fig. 141, except that the core-wall intersects the upstream face of the dam at about the water line. It will be noted that, although the line of saturation has the same slope as that in Fig. 141, it is much lower. As the proportion of the cross-section that is saturated is much less in Fig. 145 than in Fig. 141, it follows that the former section is much more stable. An additional advantage claimed for the arrangement shown in Fig. 145 is that the part of the core-wall above the intersection with the upstream face may be used, in many cases, in lieu of rip-rap, to break the force of waves and protect the slope. Wave action, of course, will erode the bank on the upstream side of the core-wall to a very flat slope, but the effect of this is limited to a few feet below the water surface. The arrangement is not so effective as a protection against wave action where there is a great variation in the elevation of the water surface. A core-wall of this kind is generally reinforced so that for a short distance it will act as a cantilever.

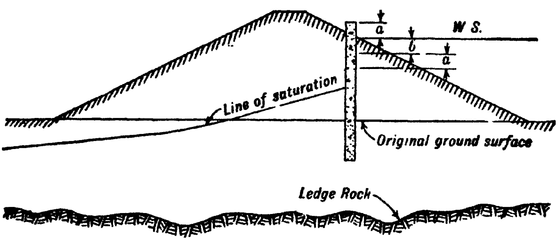

FIG. 145.—Showing the Advantage of Placing the Core-wall Upstream from the Centerline of the Dam.

Determination of the Location of the Line of Saturation.—The foregoing cases serve to illustrate certain general principles in the design of earth dams. They particularly emphasize the fact that the position of the line of saturation is an important indication of the relative safety of the structure. It is not essential for the engineer to be able to predict the exact position of the line of saturation in a dam designed for some particular site; but it is desirable that he be able to predict, with reasonable accuracy, the limiting positions between which the line of saturation will be found.

Under certain conditions it is possible to predict, by a method using the known laws governing the movement of underground waters, the approximate

position of the line of saturation.[2] However, in many cases its location must be determined from practical experience based on a knowledge of the action of seepage, as previously described.

Drainage of Earth Dams.—In many earth dams the downstream part is drained artificially in order to lower the line of saturation and thus prevent the saturation of the downstream toe. If the foundation is relatively impervious, as compared with the material of the dam, an artificial drainage system is especially applicable. If an earth dam is composed of fine sand and the foundation is of clay or rock, any water that finds its way into the comparatively porous dam cannot drain away through the soil of the foundation, and, consequently, the downstream toe will be saturated, causing a dangerously unstable condition of the downstream part of the bank.

In most cases of this kind a core-wall should be used, but even then artificial drainage may be necessary. In most instances where the foundation soil is of insufficient depth to provide natural drainage, or is too impervious to provide the necessary natural drainage, there should be a system of artificial drainage. Such drainage is often provided by digging trenches, several feet deep, perpendicular to the axis of the dam, which may be filled with broken stone, large one-man stones being placed on the bottom and the sizes decreasing toward the top, where fine crushed stone or gravel is used. Terra cotta sewer pipe may be laid in the trench, with open joints, and surrounded with crushed stone or gravel.

Drainage trenches should usually be perpendicular to the axis of the dam, but in some cases they should follow the contour of the ground. Several laterals, feeding the main drain, may be used to advantage. Such laterals generally consist of lines of 6- or 8-in. terra cotta sewer pipe laid, with open joints, directly on the ground. The joints should be amply protected by gravel or crushed stone.

The drains should not be laid too far apart, and, if drainage is to be effective, the distance between the main drains should not be more than one-fourth the thickness of the dam. The size and spacing of the drains and the quantity of water that they will collect may be predicted by the methods already outlined.

The drains should have an unobstructed outlet below the downstream toe. As a part of such a system of drainage, heavy rock fills are often placed at the downstream toe of an earth dam. If rock is available, it is good practice to place a rock fill at the toe, as the stability of the structure is thus greatly increased, and the danger from sloughing is lessened. The maximum width of the base of such a rock fill should not, in general, exceed one-third the total thickness of the base of the dam.

Frequently a rock toe and drains are both provided; but in such cases the rock toe is virtually a spoil bank from excavations for other structures. Unless the rock toe is relatively large, it cannot take the place of adequate drains.

112. Criterion 3.—Criterion 3 states that " the upstream and downstream

[2] "The Design of Earth Dams," by Joel D. Justin, Trans. Am. Soc. C. E., Vol. LXXXVI.

slopes must be such that, with the materials used in the construction, they will be stable under all conditions."

Slope of the Upstream Face.—The slope of the upstream face of the dam should generally be determined by the under-water angle of repose of the material. This angle, usually, will be flatter than the angle of repose for the same material in air. Other things being equal, a material having a high unit weight will stand on a steeper slope under water than one with a low unit weight. The slope of the upstream face should be flatter than that indicated by the experiments as safe.

In general, the slope of the upstream face, or that part of it which is to be under water, should not be steeper than one-half that at which the same material would stand out of water.

The upstream slope of some earth dams has been made as steep as 1 on 1½, but the writer believes that such steep slopes should be used only with the heaviest and most stable materials, and even then they should not be used in earth embankments more than 15 ft. high. A slope that is to be exposed to the action of water should generally not be steeper than 1 on 2.

This question of slopes is also quite largely one of superimposed loads. As a saturated material will support less load than a dry material, the upstream face must often be made flatter than the downstream. As greater depths are reached, the loads become greater and greater and the base must widen out to carry it. Thus the slopes of a very high earth dam are usually concave, beginning at the top with fairly steep slopes and changing to flatter and flatter slopes as the bottom is approached.

Slope of the Downstream Face.—The slope of the downstream face will generally be determined by Criterion 2, which requires the line of saturation to intersect the base at a point well within the downstream toe. The slope of the downstream face, however, should be flatter than the angle of repose of the material of that part of the dam, even if the position of the line of saturation would permit the use of a steeper slope.

Berms.—Rain will seep into coarse-sand dams as fast as it falls. For dams composed of less pervious materials, some provision must be made to prevent erosion from surface runoff. For such cases, berms from 6 to 20 ft. wide are common for dams more than about 30 ft. high; but it is difficult to justify a berm wider than that required to collect the runoff. The spacing of berms in high dams is fixed, of course, by the relative porosity of the materials in the dam. A spacing of about 30 ft. difference in elevation is common for quite impervious materials.

The outer edge of berms should be higher than the inner edge, to prevent rain-water from flowing over the edge and down the slope. A paved gutter should be placed at the inner edge of the berm to conduct the storm-water to the side of the valley, where other gutters conduct it to the toe of the dam. In many of the largest and highest earth dams, the storm-water from the berms is collected by catch-basins and conducted through storm sewers to the main drainage system at the downstream toe of the dam.

On some dams, berms are built on the upstream face if the pond is to be empty for long periods, and a berm should always be used as a shoulder against

which to build the bottom of the rip-rapping or concrete paving used as a protection against wave action.

Blankets.—A blanket of fine material is sometimes spread over the bottom of the reservoir and a part of the upstream face, to reduce percolation to a minimum and to prevent piping through a porous foundation. Such a blanket is, under some conditions, both cheaper and more efficacious than sheet piling or a core-wall. Figure 146 shows a cross-section of an earth dam in which such a blanket has been used. The fear has sometimes been expressed that fill of such fine material would be carried away, percolating through the coarser material. However, tests conducted by the late Frederic P. Stearns [3] prove that there is absolutely no practical danger of this.

Protection of Top and Downstream Face.—A dense growth of vegetation assists materially in the protection of the surface from erosion due to rain and wind. For dams composed of easily eroded, relatively impervious materials, it is advisable to cover the surface with 8 or 10 in. of rich top soil which should be treated with about 600 lb. of good fertilizer per acre. The surface should then be freshly raked and seeded to grass. A good seed mixture for

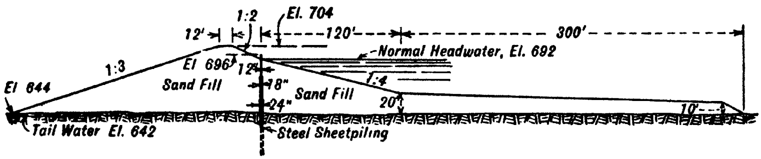

FIG. 146.—Cross-section of Junction Dam, Mainstee River, Michigan, Showing Upstream Blanket Fill of Fine Material.*

* Designed and built under supervision of Fargo Engineering Co. of Jackson, Mich. From Engineering News-Record, Vol. 81, page 493.

this purpose is Red top, 12 lb., white clover, 6 lb., and Canadian blue grass, 10 lb. per acre. Recleaned seed should be used. In the South, Bermudez grass and " wire grass " have been used with great success.

Charles H. Paul states that Japanese honeysuckle and sweet clover were successfully used for the protection of the downstream slopes of the dams of the Miami Conservancy District in Ohio, under conditions which were so arid that grass could not be successfully used.[4]

Matrimony vine (*Lycium vulgare*) has been successfully used for bank protection along the Susquehanna River at Harrisburg, Pa., and also in protecting the slopes of the Wildwood Reservoir embankment, near Harrisburg. The shoots of this vine are planted, and the vine grows along the ground, sending out roots which enter the ground for some depth and then start other shoots. The vine, which was brought from the Mediterranean, is very hardy. In a few seasons, the bank presents the appearance of a continuous tangle of vines, and the top soil becomes a tangle of roots. Banks thus protected have

[3] Trans. Am. Soc. C. E., Vol. XLVIII, page 267.

[4] For further description of experiments on grass planting for dams see " Tests on Growing Vegetation on Gravel Dams," by C. H. Eiffert, Eng. News-Record, Vol. LXXXVII, p. 55.

withstood successfully occasional wave and stream action, but, of course, could not be depended on to resist continuous wave action.

Protection of Upstream Slope.—It is necessary, on many dams, to protect the upstream face from wave action. A discussion of the height of waves, and the height to which they will ride up on the embankment, is given in Sec. 115. The depth of the protection depends on the depth to which the reservoir is likely to be drawn down.

Log booms placed in front of the slope have proved to be effective in breaking the waves before they strike the slopes. For many years the Spring Valley Water Company, of San Francisco, Calif., has protected the earth slopes of its reservoirs by anchoring booms about 3 ft. from the upstream slope.

In reservoirs that are emptied frequently, the bank protection should begin at the upstream toe. If it can be determined that the reservoir will be drawn down below a certain elevation only once in a term of years, it is sound engineering not to carry the bank protection much below that elevation, at which point a berm should be provided. The function of this berm is to make a substantial shoulder to serve as a support for the rip-rap.

There is no material superior to rip-rap for the protection of the upstream face of an earth dam from wave action. Rip-rap is of two classes, random and hand-placed. The former consists of stones dumped in place from cars or wagons or tossed in place by hand. If random rip-rap is dumped in place from cars, the individual stones may be any size up to the capacity of the steam shovel. Such rip-rap is sometimes termed cyclopean rip-rap.

Hand-placed rip-rap consists of one-man stones laid on edge on a bed that has been prepared and graded. An effort is made to break joints as much as possible, and the voids are filled with smaller stones. Hand-placed rip-rap is generally from 12 to 18 in. thick, the minimum size of individual stones frequently being specified as 12 by 12 by 3 in.

The best hand-placed rip-rap approaches good dry rubble in quality and appearance. The bottom of the band of rip-rap on the upstream face of an earth dam should rest on a shoulder or berm in the embankment; otherwise, the weight of the rip-rap might be sufficient to cause it to slide down the saturated upstream face. The bottom course of the rip-rap should be formed with headers composed of stones twice as deep as the other stones, and set into the bank in a trench. This aids in holding the upper courses in place.

If the material forming the embankment is very impervious, like clay, there should be a layer of gravel to form the bed on which the rip-rap is laid, as otherwise, when the reservoir is drawn down, the pressure of the water in the fill may cause the rip-rap to heave and slide. The layer of gravel performs the function of a drain. The same, of course, applies to a concrete paving of blocks.

A concrete lining is sometimes used on the upstream face of earth dams. It is generally poor practice to place much reliance on a concrete pavement for keeping water from entering the embankment. The function of such a lining, in the writer's opinion, is wave protection. Sometimes the concrete lining is built as a reinforced monolith, the percentage of steel being about 0.3 per cent

of the effective area of the cross-section of concrete. The writer believes that such monolithic linings are, in most cases, of questionable value. When the bank settles, the reinforced concrete lining will be left unsupported at some points, at which the slab will crack, and wave action may eventually break it up, allowing the waves to enter the gaps and work disastrous effect.

It is best to make the concrete lining of square blocks, generally not larger than 6 by 6 ft. It is not usually necessary to reinforce these blocks. The thickness of the block in inches should be the same as the dimension of the block in feet, that is, a block 6 ft. square should be 6 in. thick. The square should be poured alternately, the blocks being separated by layers of 3-ply tar paper, so that they will adjust themselves to the surface of the embankment in case of settlement. Sometimes, pre-cast concrete slabs of much smaller size are used. Small blocks of wood, ½ in. thick, may be used to separate the slabs, thus insuring their settlement with the embankment and preventing bridging. Parallel to the center line of the embankment a concrete curb should be built against which to place the concrete lining. This curb should be at the inner edge of a berm or shoulder in the embankment and should extend not less than 18 in. below the bottom of the concrete lining.

If the concrete paving is monolithic, or if the concrete blocks are set without spaces between them, it is essential to provide numerous weep-holes through the concrete, in order to allow the water in the embankment to drain away when the reservoir is drawn down quickly. Otherwise, hydrostatic pressure behind the concrete lining might break it or cause it to slide down the slope, and possibly, also, cause the sloughing of some of the saturated embankment material.

The use of concrete or rip-rap for protection against wave action should depend on the relative cost of the two materials at the site. A concrete lining, however, should never be credited with performing any of the functions of a core-wall, as such an assumption is a dangerous fallacy.

The slope that will withstand wave action successfully, without protection, depends on the nature of the material composing it, and the height of the waves.

Heavy materials, containing from 5 to 10 per cent of gravel and stones on a slope of 1 on 3, will, with any ordinary reach of reservoir, withstand wave action quite successfully. Unless the reach of the wind on the surface of the reservoir is small, fine sand and clay require such flat slopes that, for reasonable safety against wave action, it is cheaper to use rip-rap or other protection.

If the material of the upstream face contains a little stone and gravel, some engineers adopt a fairly flat slope and use no rip-rap or other bank protection at the time the dam is built, with the understanding that the cost of such protection is merely being deferred. If the embankment withstands the wave action successfully, the expenditure for rip-rap may be saved or postponed.

Considerable danger is involved in this course, because after the engineer has turned the works over to the owners, serious erosion of the embankment may take place. The operating officials may not realize the importance of

protection, and action may be postponed until an alarming conditions exists and the partial or total failure of the embankment results.

Core-wall to Protect Upstream Face against Wave Action.—If the core-wall is placed so that it intersects the upstream face of the dam about at the elevation of normal head-water, as in Fig. 145, it may frequently be used to protect the embankment against wave action. In Fig. 145 the distance, a, from the elevation of normal head-water to the top of the core-wall, should be somewhat more than the highest waves to be expected. The pond, it is assumed, is never to be drawn down below normal head-water elevation more than the predetermined distance, b. Then, when the reservoir is drawn down to minimum elevation, wave action may wear the slope down to an elevation equal to head-water elevation minus $(a + b)$. If the wall is sufficiently strong to support the bank for the distance $(a + b)$, rip-rapping of the upstream slope may be omitted. After the upstream face has been worn down to the dotted line shown, the core-wall will be effective in protecting the remainder of the embankment.

Core-walls projecting above the dam, as shown in Fig. 145, have been used in many power dams throughout the Middle West. At the Wissota hydroelectric project, near Chippewa Falls, Wis., this procedure saved about $60,000 worth of rip-rap.

Other advantages of placing the core-wall in this position in the cross-section are the increase in stability and the fact that the line of saturation is thus caused to fall farther inside the downstream toe.

Stability of the Foundation.—Upstream and downstream slopes that might otherwise be satisfactory became unstable on an earth dam having an unstable foundation. It is sometimes necessary to flatten both the upstream and downstream slopes in order to obtain increased bearing area. A foundation soil that is stable and capable of sustaining a great load in its natural condition, may become very unstable when it becomes saturated after the dam is placed in service, and cause the sliding and subsidence of the slopes.

When the reservoir is full, the effective weight of the material in the saturated part of the dam is decreased. As the downstream part of the foundation is drained more or less, a slide is more likely to occur on the upstream face when the reservoir is only partly filled.

113. Criterion 4.—Criterion 4, for the design of earth dams, states that, "there must be no opportunity for the free passage of water from the upstream to the downstream face," which means the passage of water, through some well-defined seam or passageway, in quantities greater than would be indicated by the term seepage. If water under pressure has access to such a passageway, particles of the embankment will be dislodged, and the opening thus enlarged may cause the destruction of the dam if prompt action is not taken.

Cause of Free Passage of Water through Dams.—Seams or openings through an earth dam may be caused in a number of ways, chief among which are the following:

(1) By water following the exterior surfaces of conduits through the embankment;

(2) By burrowing animals, such as muskrats;
(3) By the placing of very pervious material, such as large stones, in an otherwise impervious embankment, in such a manner as to make a blind drain from the upstream to the downstream face;
(4) By failure to bond and compact the succeeding layers of the embankment properly;
(5) By the use of a layer of very pervious material over a layer of impervious material;
(6) By failure to bond the lower layers of an earth dam properly to the foundation;
(7) By water following the smooth surfaces of concrete abutments or other concrete structures.

Conduits through the Dam.—Conduits through an earth dam should always be placed on or below the original surface, both because settlement is thereby reduced and because it is very difficult to place a tight fill under the conduit.

As water has a tendency to follow along smooth surfaces, ample cut-offs of concrete should be provided along the conduit to interrupt the continuity of surface. The conduit should preferably be built in an excavated trench. In such cases, the necessary cut-offs should be placed in lateral trenches extending to each side and under the conduit and completely filled with concrete.

Pressure conduits should not be used through earth dams unless absolutely necessary. Leaks due to settlement, earthquakes, or weakness may result in a concentrated flow in the downstream part of the dam and cause failure. If such structures must be provided, they should be of an absolutely permanent nature, very strong, and with very stable foundations. In some cases, pressure conduits are carried in a tunnel around the dam.

If conduits, for emptying the reservoir, pass through the dam, it is better to have the control gates at the upper end so that there will be no great pressure in them under the dam.

Protection against Burrowing Animals.—If a core-wall extending above high-water mark is included in the design, there is nothing to fear from burrowing animals. If the upstream face is protected with rip-rap or with concrete slabs, they will have no opportunity to injure the embankment. If the material of the embankment is compacted thoroughly, the muskrat will go elsewhere in search of a home. The free-board and top width of high earth dams are generally so great that the muskrat does not burrow through to the downstream face.

Accidental Blind Drain through Embankment.—The careless placing of materials occasionally causes what is, in effect, a blind drain, from the upstream to the downstream face. Such a condition is avoided by preventing coarse material from forming a continuous line from the upstream to the downstream face. If the central pool of a hydraulic-fill dam is narrow, there is a tendency for tongues of coarse material to be projected across the pond. This is often combated by breaking up such tongues with plows.

Passage of Water through Uncompacted Material.—If succeeding layers of the embankment are not properly compacted and bonded, there is a great chance for water to pass through the loose material. After work on an embankment has been suspended for a time, the surface may become hard and smooth. If additional layers are placed, without special precautions to secure a bond between the new work and the old, there will be danger of a leak along the plane of contact with the pervious uncompacted layer. The surface of the hardened layer should be scarified and moistened before the new layer is placed.

Passageway for Water through Pervious Layer.—A comparatively free passageway for water is sometimes provided through the dam by a pervious layer lying between two impervious layers. Under such conditions the pervious layer forms a pipe, as it were, and water under a head flows through the layer to the downstream face with sufficient velocity to move some of the particles of the material. Such layers in the upstream part of the dam should be prevented by rejecting such material from the borrow-pits for use on the upstream side of the dam and diverting it to the downstream side.

When an embankment is built by using trains of dump cars running on track laid directly on the embankment, it is no simple matter to place the tight material on the upstream part of the embankment, and the pervious material on the downstream part. If the character of the material varies greatly, it is a source of great annoyance to the engineer and expense to the contractor.

Core-wall to Prevent Passage of Water through Dam.—The use of a substantial core-wall on a proper foundation will prevent the free passage of water through the embankment. It is necessary, however, to observe the various precautions outlined herein, and to consider the use of a core-wall merely as a means of making assurance doubly sure.

Cut-off Buttresses on Spillway and Power-house Walls.—Spillway abutments, power-house walls, or other concrete walls extending through the dam in an up and downstream direction should be provided with cut-off buttresses projecting well into the embankment. There should be at least two buttresses on each wall passing through the embankment. It is unnecessary to make these buttresses more than 12 in. thick, if they serve only as cut-offs, but they should be lightly reinforced in order to prevent their separation from the main wall. The cut-off walls and the rear face of the main wall should have a slight batter (about 1 on 20), so that shrinkage of the embankment will tend to bring the earth in closer contact with the concrete.

114. Criterion 5.—This criterion requires that the water that passes through or under the dam must have a velocity so small that there will be no undue waste of water and that the water will be incapable of lifting any of the material of the foundation when it rises to the surface below the toe. The latter phenomenon is called "piping."

C. S. Slichter's formula for flow through soils, applied to this case, is:

$$Q = \frac{Kha}{\quad} \qquad (109)$$

where Q = the discharge in cubic feet per minute;
a = the cross-sectional area, in square feet, of the material through which the discharge is taking place;
K = transmission constant of the material;
h = the head on the dam, i.e., the difference between the elevations of head and tail-water, in feet; and
l = the length of the path of percolation, in feet.

But the actual velocity, in feet per minute, between the particles composing the material, is:

$$v = \frac{Q}{aP}, \qquad (110)$$

where P is the porosity of the material, expressed as a decimal.

Therefore, combining Eqs. (109) and (110), we have:

$$v = \frac{hK}{lP} \qquad (111)$$

In order to determine the velocity for any length of path of percolation and head on the dam, it is only necessary to know the porosity, P, and the transmission constant, K.

The porosity may be determined as follows: Determine the specific gravity of dry particles of a sample, by the usual method commonly employed for Portland cement.

Let S = specific gravity of the particles;
W_1 = weight of the sample, dry;
W_2 = weight of the sample when saturated with water;
P = porosity, or percentage of voids expressed as a whole number.

Then

$$P = \frac{100(W_2 - W_1)}{\frac{W_1}{S} + (W_2 - W_1)}. \qquad (112)$$

The transmission constant is indicated in Table XXXIV [5] for soils of various effective sizes and degrees of porosity, at 60° F., as computed by Professor Slichter. Table XXXV is a correction table which gives the multipliers to use with the transmission constants derived from Table XXXIV, in order to obtain the transmission constants for other temperatures.

The effective size of the material may be used in the first column of Table XXXIV. The " effective size " may be determined by sieve analysis and is that diameter, in millimeters, such that 10 per cent by weight of the particles are smaller and 90 per cent are larger than it. The relative fineness of soils, when used in percolation studies, is usually expressed in terms of their effective size.

[5] From Water Supply Paper No. 140, U. S. Geol. Survey.

TABLE XXXIV

Transmission Constants from which the Velocity of Water in Sands of Various Effective Sizes of Grain can be Obtained

Table Computed for Temperature of 60° F.; Results for Other Temperatures can be Found by the Use of Table XXXV

Diameter of Soil Grains, in Millimeters	Porosity						Kind of Soil
	30%	32%	34%	36%	38%	40%	
0.01	0.000033	0.000040	0.000050	0.000060	0.000072	0.000085	Silt
0.02	0.000131	0.000162	0.000198	0.000239	0.000286	0.000339	
0.03	0.000296	0.000364	0.0004460	0.000538	0.000645	0.000763	
0.04	0.000527	0.000648	0.0007940	0.000958	0.001145	0.001355	
0.05	0.000822	0.001012	0.001240	0.001495	0.001790	0.002120	Very fine sand
0.06	0.001182	0.001458	0.001784	0.002150	0.002580	0.003050	
0.07	0.001610	0.001983	0.002430	0.002930	0.003510	0.004155	
0.08	0.002105	0.002590	0.003175	0.003825	0.004585	0.005425	
0.09	0.002660	0.003280	0.004018	0.004845	0.005800	0.006860	
0.10	0.003282	0.004050	0.004960	0.005980	0.007170	0.008480	Fine sand
0.12	0.004725	0.005830	0.007130	0.008620	0.01032	0.01220	
0.14	0.006430	0.007940	0.009720	0.01172	0.01404	0.01662	
0.15	0.007390	0.009120	0.01115	0.01345	0.01611	0.01910	
0.16	0.008410	0.01036	0.01268	0.01531	0.01835	0.02170	
0.18	0.01064	0.01311	0.01605	0.01940	0.02320	0.02745	
0.20	0.01315	0.01620	0.01983	0.02390	0.02865	0.03390	
0.25	0.02050	0.02530	0.03100	0.03740	0.04480	0.05300	Medium sand
0.30	0.02960	0.03640	0.04460	0.05380	0.06450	0.07630	
0.35	0.04025	0.04960	0.06075	0.07330	0.08790	0.01039	
0.40	0.05270	0.06480	0.07940	0.09575	0.1145	0.1355	
0.45	0.06650	0.08200	0.1005	0.1211	0.1450	0.1718	
0.50	0.08220	0.1012	0.1240	0.1495	0.1780	0.2120	Coarse sand
0.55	0.09940	0.1225	0.1500	0.1810	0.2165	0.2565	
0.60	0.1182	0.1458	0.1784	0.2150	0.2580	0.3050	
0.65	0.1390	0.1710	0.2095	0.2530	0.3030	0.3580	
0.70	0.1610	0.1983	0.2430	0.2930	0.3510	0.4155	
0.75	0.1850	0.2278	0.2785	0.3365	0.4030	0.4770	
0.80	0.2105	0.2590	0.3175	0.3825	0.4585	0.5425	
0.85	0.2375	0.2925	0.3580	0.4325	0.5175	0.6125	
0.90	0.2660	0.3280	0.4018	0.4845	0.5800	0.6860	
0.95	0.2965	0.3650	0.4470	0.5400	0.6460	0.7650	
1.00	0.3282	0.4050	0.4960	0.5980	0.7170	0.8480	Fine gravel
2.00	1.315	1.620	1.983	2.390	2.865	3.390	
3.00	2.960	3.640	4.460	5.380	6.450	7.630	
4.00	5.270	6.480	7.940	9.575	11.45	13.55	
5.00	8.220	10.12	12.40	14.95	17.90	21.20	

Given a dam having the following characteristics:

$$h = 20 \text{ ft.};$$
$$P = 0.34;$$
$$\text{Effective size} = 0.5 \text{ mm.};$$
$$l = 100 \text{ ft.};$$
$$\text{Temperature} = 70° \text{ F.};$$

From Table XXXIV, the coefficient, K, for the given value of P and the

TABLE XXXV

VARIATION, WITH THE TEMPERATURE, OF THE FLOW OF WATER OF VARIOUS TEMPERATURES THROUGH A SAND, 60° F., BEING TAKEN AS THE STANDARD TEMPERATURE

Temperature, in Degrees Fahrenheit	Relative Flow	Temperature, in Degrees Fahrenheit	Relative Flow
32	0.64	70	1.15
35	0.67	75	1.23
40	0.73	80	1.30
45	0.80	85	1.39
50	0.86	90	1.47
55	0.93	95	1.55
60	1.00	100	1.64
65	1.08		

effective size, is 0.1240. Therefore, from Eq. (4), the actual velocity, for 60° temperature, is:

$$v = \frac{20 \times 0.124}{100 \times 0.34} = 0.073 \text{ ft. per minute.}$$

For 70° temperature, we have, from Table XXXV,

$$v = 0.073 \times 1.15 = 0.084.$$

Justin has shown [6] that the dam will be safe against piping action if the actual velocity of flow under the dam does not exceed 0.5 ft. per minute for fine silt or coarser material. Clay would not be safe against this velocity, but the velocity through clay is negligible. Assuming a moderate-sized dam having a length of 100 ft. and a depth of permeable material in the foundations of 20 ft., if the porosity of the material in the foundations is 0.34, the total net area of waterway under the dam is 0.34 × 100 × 20 = 1360 sq. ft. and the discharge is 1360 × 0.5/60 = 11.3 cu. ft. per second, which is excessive. Therefore, it would seem that a dam that is sufficiently tight to prevent undue waste of water would be safe against piping. However, the length of the path of percolation should be investigated for both.

The length, l, of the path of percolation is ordinarily the length of the base of the dam; but the length of the path may be increased by providing a cut-off wall of sheet piling, puddle, or masonry, or by providing an impervious blanket on the bottom of the reservoir, upstream from the dam. In a dam with a cut-off wall, the length of the path should be measured in a straight line from the upstream toe or upper end of the blanket to the bottom of the cut-off wall, and thence in a straight line to the downstream toe.

115. Criterion 6.—Criterion 6 states that "the free-board must be such that there is no danger of overtopping by wave action." The definition of "free-board," as here used, is, "the difference in elevation between the top of the dam and the spillway crest."

The top width of earth dams is a factor in the protection of the structure

[6] The Design of Earth Dams, Trans. Am. Soc. C. E., Vol. LXXXVII (1924).

against wave action, as a wide top width is much safer against wave action than a narrow one. Therefore, it is more logical to proportion the top width to the probable height of waves than to the height of the dam. However, it is more economical to provide a greater height of free-board than a wider top, and it is quite undesirable to have the waves reach the top of the dam, even with a very wide top width. Except for very small dams, the top width should be made at least 10 ft. in order to provide a work space for maintenance or repairs, or a roadway. It is impossible to formulate a general rule for top width, because, with ample free-board, there is no logical reason for a width in excess of about 10 ft., although past practice has established the precedent of a top width between 20 and 30 per cent of the height of the dam or a similar rule.

The height of free-board should be the sum of the maximum rise of water surface in the pond above the spillway crest during the greatest probable flood, the height to which waves will ride up on the slope of the dam, and the maximum depth of frost, provided ample margin of safety is contained in the determination of these three factors.

The height to which water may rise in the pond is discussed in Chapter V and in Sec. 74.

The height to which waves will ride up on the embankment is not known exactly. They will ride higher on a flat than on a steep embankment. Stephenson gives the following equation for determining the height of waves in reservoirs:

$$H = 1.5\sqrt{D} + 2.5 - \sqrt[4]{D}. \qquad (113)$$

Where H = the height of the waves, from trough to crest, in feet, and D = the exposure, or "fetch," in miles. The actual height of the waves above mean water surface is about one-half that given in the equation and, if we make the reasonable assumption that the water will ride up the embankment to a vertical height not exceeding twice the height of the wave above mean water, we may use the value of H for the free-board allowance for wave action.

The depth to which frost action will penetrate at the top of an earth dam depends upon the nature of the materials and upon climatic conditions. The top of the dam is in a position to drain well and, if it is composed largely of sand or gravel, little or no frost action need be considered.

Too much stress cannot be placed upon the necessity of providing an ample margin of safety in the determination of the free-board. Most failures of earth dams result from overtopping during floods. It is good practice to provide a safety device to guard against unforeseen conditions. In the case of a dam having as an auxiliary a number of low dikes, one or more of the dikes may be provided with a lower top elevation on the assumption that, if higher water than is anticipated should occur, the smaller structure would fail and provide an additional outlet for the flood waters, with relatively little damage. Semi-permanent sluices, which may be blown up for the same purpose, have also been provided in concrete spillways for earth dams. Such devices are particularly applicable to streams for which there are no gagings from which the flood characteristics may be predicted.

116. Preparing the Site.—Trees, stumps, and sod should be removed from the site of the dam. All roots of any size should be grubbed out and removed, and even small ones, if present in bunches, should be taken out. Generally the top soil below the sod should be excavated, particularly if it is high in vegetable matter or otherwise not suitable for a dam foundation, or if it will not bond with the material composing the dam. This applies particularly to the upper two-thirds of the base, although frequently the contractor is permitted to spoil this material in the extreme upper toe of the dam.

It is a matter of prime importance that there be no definite dividing plane between the dam and its foundation. The surface of the completed foundation should be plowed up and dragged longitudinally with a disk harrow, just before the embankment is placed. The surface should then be liberally moistened, if a rolled embankment is to be used, so that the first layer of the embankment will be forced down into the foundation and leave no dividing plane.

The foundation should be completed before the excavation for the core-wall or core-trench is started, so that the original surface will not be covered by the excavated material.

Where the material immediately underlying the proposed dam is considered too pervious, a cut-off trench is excavated to a more impervious stratum. Should this trench be for the reception of a core-wall, it may be sheeted; but, if it is for a dam without a core-wall, it should have side slopes which are perfectly stable, particularly if the dam is to be built by the hydraulic-fill method, for which the trench will be filled with water before the depositing of the material for the dam is started. The width of the bottom of the trench depends, of course, upon the nature of the materials in the foundation and in the center of the dam.

117. Segregation of Materials.—The ideal dam consists of a relatively thin, impervious barrier to the passage of water, supported by coarser materials. The impervious barrier, theoretically, belongs on the water side of the dam, to prevent the water from seeping into the dam; but, practically, the use of a core-wall or a puddled core necessarily brings the barrier near the center of the dam.

In a rolled embankment, the finest materials should be deposited in the upstream third of the dam, and the coarsest in the downstream third. This procedure is practicable only if the material in one borrow pit is much coarser than that in another, and such a selection can be readily made. In a hydraulic-fill dam, the process automatically deposits the finer materials in the center of the dam, with the coarser materials on each side of the center.

118. Core-walls.—Before the use of concrete became so general, it was usual for the core-wall in the center of the dam to be made of puddle, consisting of a mixture of gravel, sand, and clay. The puddle core-wall is still used extensively in some foreign countries; but it has been practically discontinued here as its cost is little different from that of a thinner concrete wall of the same effectiveness. The puddle resembles concrete in very many respects, even to its being mixed in a "pug-mill," a machine similar to the ordinary continuous concrete mixer, the sole difference being that clay is used instead

of cement. Fanning recommends 1.0 part of clay to 2.5 parts of sand and fine gravel, to 5.0 parts of coarse gravel.

A puddled core, in this country, has now come to mean some clayey material, or sandy material high in clay, deposited through a pool of water. It is not good practice to puddle material that is very high in clay. Clay will sometimes take up 2.5 times its own weight of water, will exert practically full hydrostatic pressure, and will not drain properly in a reasonable period of time. Moreover, if allowed to dry, it will contract and leave large cracks which offer a passage for the flow of water.

The usual type of core-walls are constructed of rubble masonry, plain or reinforced concrete, timber, or, in a few cases, steel.

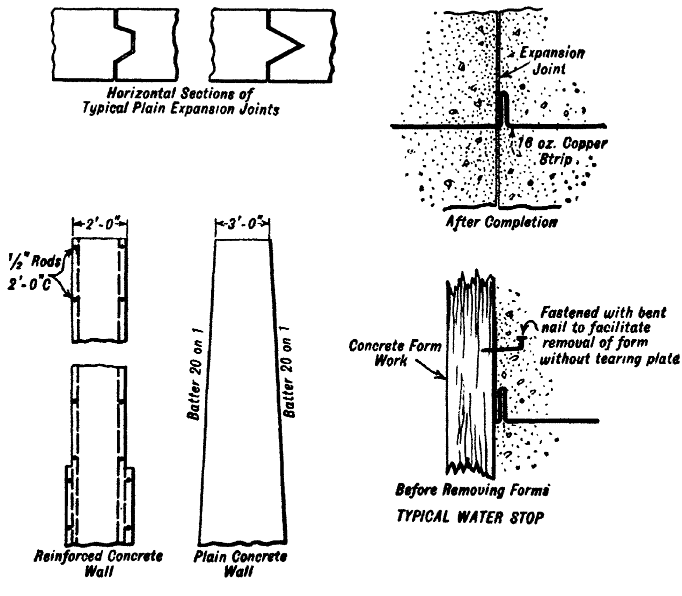

FIG. 147.—Typical Concrete Core-walls, Cut-offs and Water Stop.

Rubble-masonry walls were formerly very much used and are still used where conditions are such as to make their use cheaper than that of concrete. Some of the rubble-masonry core-walls exceeded 20 ft. in thickness at the base. Concrete core-walls are of two types; mass concrete core-walls, and reinforced concrete core-walls. The mass concrete core-walls are generally from 3 to 6 ft. wide on top, increasing in thickness as the depth below the top of the dam increases. The batter is usually the same on both sides. The reinforced concrete core-walls are from 1 ft. to 3 ft. thick and usually either batter out slightly as the depth below the top increases or else step out at intervals of 10 to 20 ft., thus making a ledge to set successive lifts of forms on. Such walls are reinforced on both sides, both horizontally and vertically, with

from 0.3 to 0.6 of 1 per cent of reinforcing steel placed near the faces. The idea is that this will produce slab action across areas of unequal pressure and will also tend to hold the wall together in case of distortion due to unequal settlement of the embankment. These thin reinforced concrete walls are sometimes spoken of as diaphragm core-walls.

Figure 147 shows details of typical concrete core-walls.

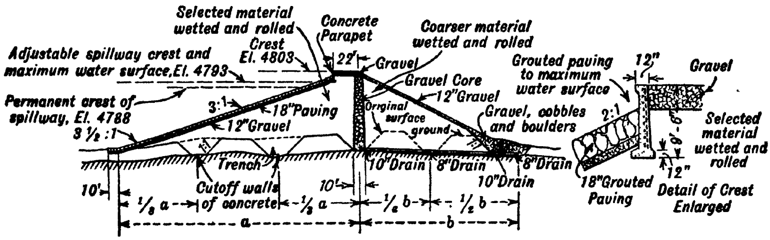

FIG. 148.—Section of Sherburne Lakes Dam, Glacier National Park, Showing Drains which Provide for Seepage Water. Built in Rolled Layers.*

* From Eng. News-Record, Vol. 80, p. 368.

Expansion joints should always be used in plain concrete core-walls. These should be provided with a groove as indicated in the figure, and should be coated with tar or similar paint to prevent adhesion of the concrete and possible cracking at other places. Additional tightness is seldom required; but, if desired, it can be obtained by the use of a water-stop as shown in the figure.

If reinforced concrete walls are not extended to an abutment of rock on

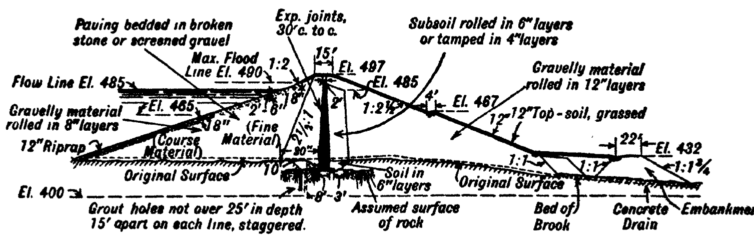

FIG. 149.—Cross-section of Phelps Brook Earth Dam for Hartford, Conn., Waterworks. Built in Rolled Layers.*

* From Engineering News-Record, Vol. 78, p. 265.

each end, or if they extend into a well-defined, completely filled rock trench, they ordinarily are not provided with expansion joints, as the reinforcement will prevent large cracks. The reinforcement should pass through the building joints. If, however, they abut at each end against a rock face or into a rock trench that is not completely filled with concrete, or if the rock face is likely to pull loose with tension in the wall, an expansion joint should be provided to prevent the possible opening up of a passage for water, which would be particularly undesirable near the junction of the dam with the rock.

The usual precautions for sealing concrete to rock, as specified for masonry dams, are particularly essential for concrete core-walls. A 1 : 2 : 4 mixture for reinforced and a 1 : 3 : 6 mixture for plain concrete walls are usual proportions.

Timber core-walls are used in some cases where it is absolutely essential that the first cost be a minimum. There is no valid objection to their use when properly built, if they remain at all times below the line of saturation. Above the lowest point reached by the line of saturation at its intersection with the core-wall, a thin reinforced concrete core-wall should be substituted for the timber section. Unfortunately, this precaution is not generally observed. If continually saturated, these timber core-walls should last many hundreds of years; and, by the time they have disintegrated, the upstream face of the dam and the bottom of the reservoir will have silted up to such a degree that a core-wall will, in all probability, no longer be necessary.

Steel core-walls are more expensive than those of wood and last no longer, if as long, as wood, below the line of saturation. Steel sheet piling has frequently been used as an extension of the core-wall into the foundation, at sites where the driving was too hard for wood piling.

119. Rolled Embankment Dams.—Unless the material is to be deposited

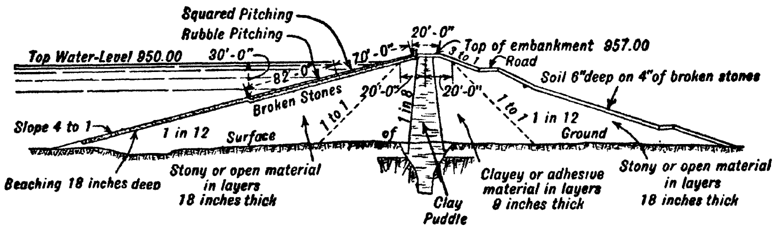

FIG. 150.—Section of the Talla Dam of Edinburgh, Scotland, Waterworks. Built in Rolled Layers.*

* From Engineering Record, Vol. 57, p. 21.

and settled by the action of water, it should be spread in thin layers and rolled. Occasionally, dams have been built by dumping the material from trestles much in the same manner that railroad fills are made. Such a method should never be used as it gives porous and unstable embankments.

The layers should never be over 12 in. in thickness and preferably very much thinner. Even in a layer 12 in. thick, the material at the top of the layer, where it receives the pressure and impact from the wheels of the roller, is much more compact than the material at the bottom of the layer. Probably the best practice with the usual run of materials is to require 6 to 8-in. layers, while materials that are readily compressible should be placed in layers 4 to 6 in. thick. If, in the case of any particular material, it is definitely determined that placing in 4-in. layers, for instance, does not secure any greater degree of compactness, then, of course, there is no use in going to the additional expense of obtaining the 4-in. layers.

The material is conveyed to and dumped on the dam by any of the various possible methods of handling dry fill, and each load is then spread by hand or

scrapers to the desired thickness. Before the foundation or any particular portion of the rolled embankment is covered with a new layer, the surface of the rolled layer should be wet with a hose or watering wagon. The wetting should be carried just ahead of the deposition of the material for the new layer. Only enough water should be used to moisten the old layer to such a degree that when the roller passes over the new layer above, the new material will be forced slightly into the old and moisture will be forced up through the new layer, leaving it slightly moist at the top. There should never be water enough used to cause the roller to slip or mire, or to give it any trouble whatever. The proper amount of water to use to produce the above effect can be readily determined after a little experience with the particular material in use. The surface of the uncompacted layer should never be wet before

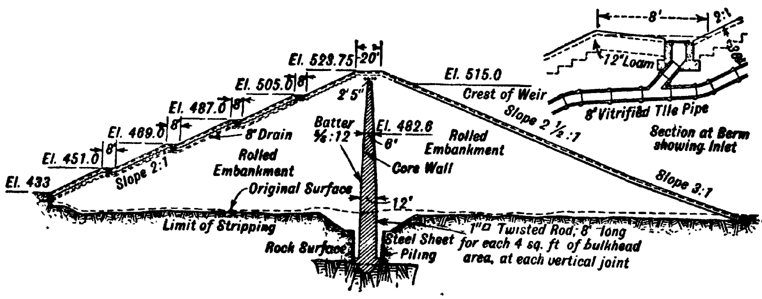

FIG. 151.—Cross-section of Morris Dam, Waterbury, Conn. Water Supply. Built in Rolled Layers.*

* From Eng. News, Vol. 65, p. 342.

rolling, as this may cause the roller to slip and make the material stick to the wheels in great " gobs."

Steam or gasoline rollers are used on most large dams. Whatever type of roller is adopted, care should be taken that sufficient concentration of pressure is being obtained. The required pressure depends upon the nature of the materials and the thickness of layers. Exploration trenches should be dug during the progress of the work to determine whether the materials, for the adopted thickness of layers and weight of roller, is being properly compacted. A relatively light roller with thin layers may prove more economical, for some materials, than a heavy roller with thick layers. A pressure of between 30 and 50 lb. per square inch, with a 1-inch penetration of the wheel, is frequently specified.

Stones higher than the thickness of the layer should be removed, and all nests of stones should be separated. The layers are deposited in an approximately horizontal position; but, for clayey materials, which are likely to become very soft with rain, provision for sloping to the faces of the dam are desirable. The importance of securing a high degree of compactness in any earth dam, whether the material is placed in rolled layers or is compacted by the action of water, is hard to exaggerate. There is a tremendous difference

in seepage through loose material and through compacted material. If the embankment is properly compacted, there will be much less settlement. Excessive settlement is dangerous to the safety of a dam.

It is often difficult to roll up close to core-walls, conduits, abutment walls, etc.; and, consequently, special pains must be taken to see that the material along these structures is properly compacted. Where the roller cannot reach, hand tamping should be done. Special care should be taken in tamping the embankment along the face of structures that pass through the dam in an up and downstream direction. The precautions that it is necessary to take in connection with the construction of such structures are outlined in Sec. 113. If the material of which the embankment is composed is at all compressible, that which is deposited against the face of these structures should be spread and thoroughly tamped in 3-in. layers. Each of these layers should be sprinkled in the same manner as the embankment which is to be rolled.

The proper distribution of fine and coarse materials in a rolled embankment has been discussed in Sec. 117. Slopes should be trimmed to a true, neat line, both for the sake of the appearance of the structure and to prevent isolated steep faces, which may slide, and additional superimposed loads due to excess materials.

120. Hydraulic-fill Dams.—A hydraulic-fill dam is an earth dam in the construction of which the materials are transported on to the dam by water and distributed to their final position by water. The conditions favorable to hydraulic-fill dams are borrow-pits at a sufficiently high level to sluice most the material by gravity into the dam, and materials that are easily eroded by water and contain only a small percentage of stones too large to be transported by this method.

The material is excavated in the borrow-pits by the use of water under heavy pressure, discharged from "giants" or "monitors," which are large nozzles, often 5 and 6 in. in diameter. The streams from the nozzles are directed against the bank of the pit, usually at a velocity of between 100 and 200 ft. per second, in such a manner that the water undercuts the bank and breaks it up. The water, carrying the loosened material, is then guided along the bottom of the pit to a sluice, consisting of a flume or pipe, through which it flows to the dam. The minimum grade of the sluice is from 3 to 6 per cent, according to the nature of the materials. If the borrow-pit is too low to sluice directly on to the dam, the sluice is made to discharge into a "hog box," or intake, from which the materials and water are repumped by dredge pump to the desired elevation. The percentage of solids in the water depends upon the nature of the materials, and particularly on the ease with which the bank can be broken down by the jet of water. As high as 20 per cent of solids has been obtained; but it is not safe to count on more than 4 to 8 per cent under the best conditions, unless experience has been had under very similar conditions. If the pits vary greatly as to coarse and fine materials, care should be taken to average the materials by sluicing from a coarse and a fine pit at the same time.

The sluices are carried across the dam, one on each side, as indicated in Fig. 152, and are made to discharge at intervals along the dam. The dis-

charge from the sluices flows toward the central pool, which is maintained, as indicated, throughout the progress of the work. The coarse material is automatically deposited on the shores of the pool, and the fine material flows into the pool with the water and is slowly precipitated. Small dikes are thrown up by hand or excavator, outside of the sluices, as indicated in the figure, to prevent the discharge from flowing over the face of the dam, and to form the outside face to true lines. The outer slopes should be carefully

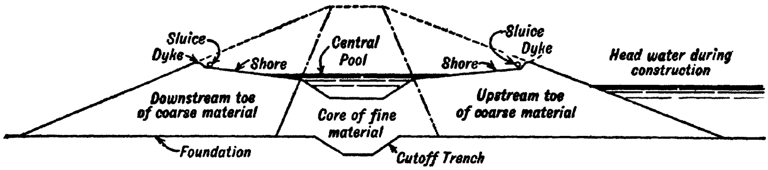

FIG. 152.—Typical Section of Hydraulic Fill Dam During Construction.

trimmed as the work progresses. The danger from slides is greatest during construction; and an unevenness of slope will produce steeper slopes at places, each with a superimposed load in the nature of excess material above it, and may cause isolated slides which might be the small start of a general movement.

The central pool is provided with two or more outlet pipes leading to the base of the dam.

Great care must be taken to fan out the discharge from the sluices to pre-

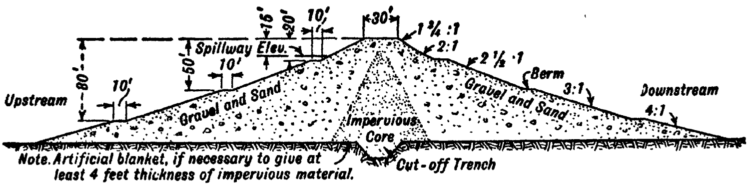

FIG. 153.—Typical Section of Earth Dams of Miami Conservancy District, built by Hydraulic Fill Method of Construction.*

* From "Core Studies in Hydraulic Fill Dams of the Miami Conservancy District," Charles H. Paul, Trans. Am. Soc. C. E., Vol. LXXXV, p. 1181.

vent it from gullying the shore and flowing with high velocity toward the central pool, carrying coarse material with it. Such high velocities are also likely to break down the side slopes of the pool and both of these actions will cause tongues of coarse material to extend into the fine core. If the core is narrow, such tongues may extend all the way through and provide a relatively free passage for seepage. This fanning out of the discharge can be accomplished by means of shear boards, which can be easily placed and adjusted to direct the flow as desired. For the same reason, if the pool, because of a stoppage of work or other cause, is allowed to get too low, it should not be filled from the sluices but from a special sluice outlet which discharges

directly into the pool without movement of materials in the dam. When the construction of the dam is first started, dikes are thrown up at both the upstream and the downstream toes to hold the central pool, which at that stage is very wide. The space between the dikes must be very carefully filled with water so as not to disturb the surface of the foundation and particularly the side slopes of the cut-off trench. The sluices are run out on these dikes and the pool narrowed to the required size by the deposition of the materials.

Short tongues of coarse material will find their way into the core despite all precautions, and too great stress cannot be placed upon the necessity for great care to limit their number and size.

The amount of very fine material that fails to be deposited in the central

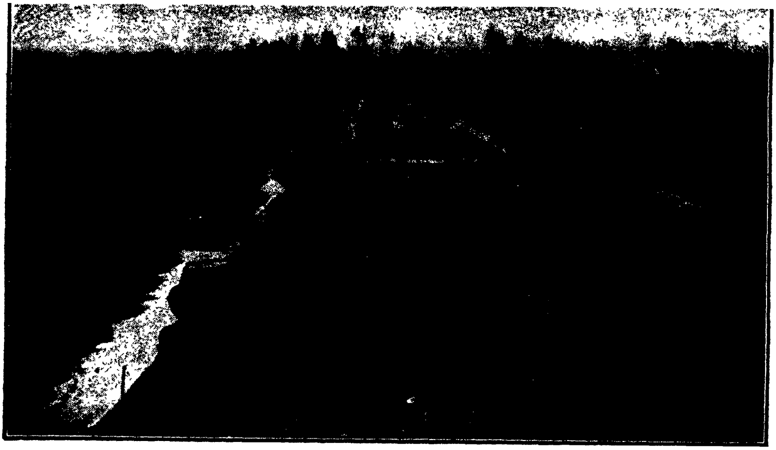

FIG. 154.—Soft Maple Hydraulic Fill Dam During Construction.

pool and is carried away through the outlet pipes, depends upon the size of the pool and therefore can be controlled by adjusting the depth, and hence the size, of the pool. Therefore, if it is considered that the material in the central core is too fine, the size of the pool can be reduced and some of the finest material wasted.

There is danger in a core composed of materials that are too fine. Very fine colloidal materials will deposit in a semi-liquid mass, will not drain readily, and may exert hydrostatic pressure on the toes of the dam and cause a disastrous slide. This feature has caused several bad failures during construction. A very good discussion of the effect of variations in the effective size of core materials is contained in "Hydraulic-fill Dams," by Allen Hazen [7] and its accompanying discussion.

If a large percentage of very fine particles is contained in the materials

[7] Trans. Am. Soc. C. E., Vol. LXXXIII, p. 1713.

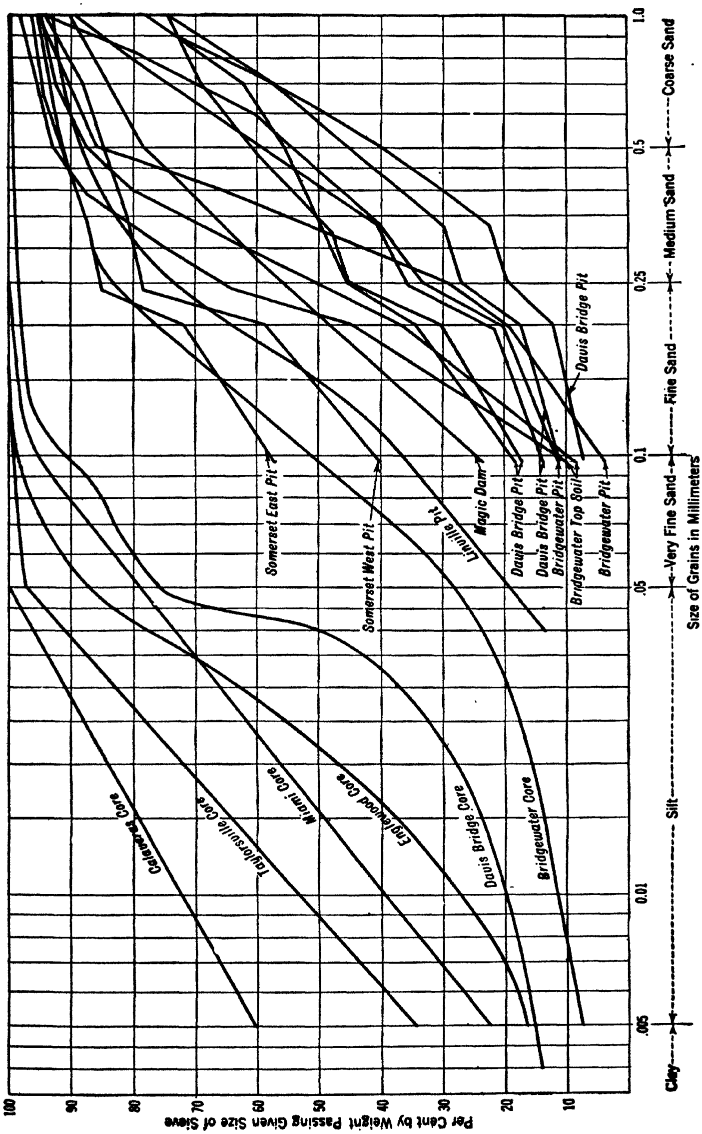

FIG. 155.—Borrow Pit Material and Resulting Cores.
(Compiled by A. S. Crane).

available for the dam, the slopes of the faces of the dam must be made quite flat, in order to provide ample mass to resist the thrust of a semi-liquid core; or else the finer materials must be wasted by reducing the size of the central pool, as previously explained. Allen Hazen suggests tentatively that an effective size of 0.01 mm. might be adopted as the ideal size for the core. Such a core would solidify promptly and would exert very little pressure on the toes. However, many successful dams have been constructed with much finer cores, even without extraordinarily flat side slopes. Fig. 155 shows the sieve analysis of the borrow-pit materials and the core materials of several

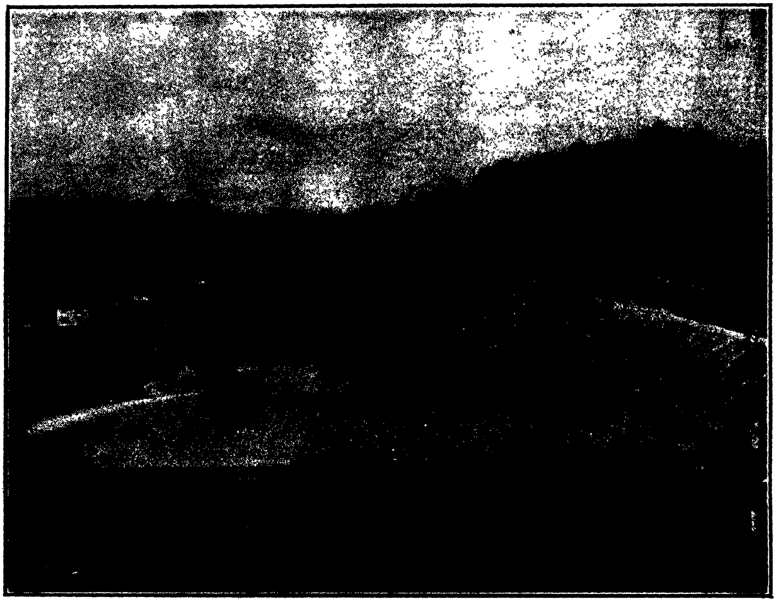

FIG. 156.—Linville Semi-Hydraulic Fill Dam of the Western Carolina Power Co. at Bridgewater, N. C.

prominent dams. The Calaveras Dam had a bad slip during construction, due to too fine core material.

The degree of fineness that is permissible, depends, of course, upon the nature of the materials. If it is pure sand without colloidal properties, it can be used with an effective size less than 0.01 mm. If, however, it has colloidal properties or has colloidal material mixed with it, care should be taken to limit its degree of fineness.

The best means of insuring the proper degree of fineness is to experiment on the material extensively before construction starts, and to examine the core frequently during construction to see that it is draining properly.

As explained heretofore, it is very difficult to prevent tongues of coarse material from projecting into the core. Hence the rejection of materials

below a given size may require a central pool, and hence a core, which is so narrow that it may be difficult, if not impossible, to prevent such tongues from projecting through the core and providing a passage for excessive seepage.

This feature, together with the amount of fines which it can be afforded to waste, sets a limit to this method of dam construction. In other words, if the materials available for the dam are too fine, it will be necessary to adopt the rolled-embankment method of construction.

121. Semi-hydraulic Fill Dams.—A semi-hydraulic fill dam is an earth dam in the construction of which the materials are transported on to the dam by some other means than water, and dumped within the section of the dam, some of this material being moved to its final position in the dam by the action of water.

The dry material is conveyed to the dam by whatever means the contractor selects, and is usually dumped from cars run out primarily on trestles as indicated in Fig. 158. Trestles 1 and 2 are first erected, and the embankment, 7, 8, and 9, deposited by shifting the track. At the same time, a nozzle, supplied by pumping equipment located on a barge in a central pool, throws a stream against the dumped material and washes the finer particles into the central pool. The location of the pool and the washing of the materials into it are similar to the action previously described for the hydraulic-fill type of construction. When the dumped embankment has progressed as far as desired toward the central pool, trestles 3 and 4 are erected and the embankment 10, 11, and 12 placed. In this manner, the dam proceeds to its completion.

Most of the precautions recommended for hydraulic-fill dams are applicable to the semi-hydraulic fill type. In this type, however, there is no opportunity for the waste of the excessively fine materials, as the water in the central pool is usually not wasted.

Often the semi-hydraulic fill method of construction is the cheapest and most convenient one that can be applied to a given site. There are, however, certain dangers generally inherent in this method of construction, which should be appreciated and guarded against if the method is adopted. Dams built by the hydraulic-fill method have been comparatively free from slides during and immediately after construction, while in the case of dams constructed by the semi-hydraulic fill method, slides during construction have occurred in a number of instances.

There seems to be a general fundamental reason for the difference in stability during construction thus indicated. The hydraulic-fill method deposits the material from the flumes near the faces of the dam; the larger particles stay there and the finer move towards the center, the finest of all going into the central pool and being deposited there. Thus the toes and faces of a dam produced by this method are pervious, allowing water to drain out from the interior of the dam.

Even if most of the drainage from the cores takes place by vertical crater action as some engineers maintain, such action takes place, not only in the portion of the core underlying the central pool, but also in those portions of the core covered by the pervious outer sections of the dam. Hence,

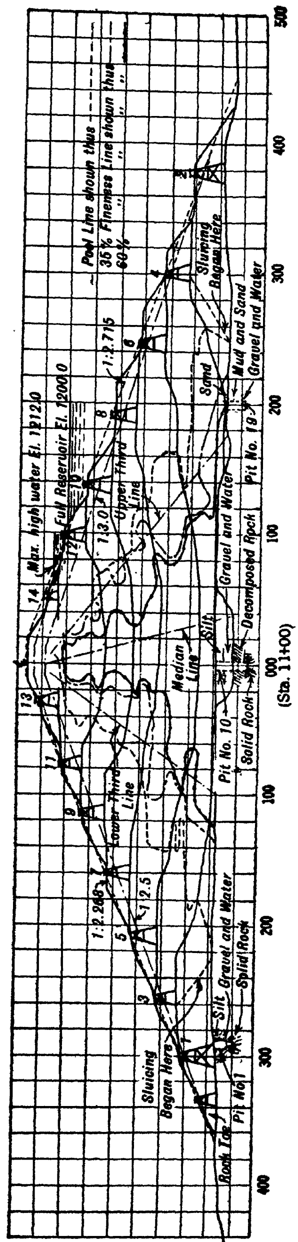

FIG. 157.—Paddy Creek Dam, North Carolina, Bridge Water Development, Built by Semi-hydraulic Fill Method.*

* From "Some Investigations and Studies in Hydraulic Fill Dam Construction," J. Albert Holmes, Trans. Am. Soc. C. E., Vol. LXXIV, p. 331.

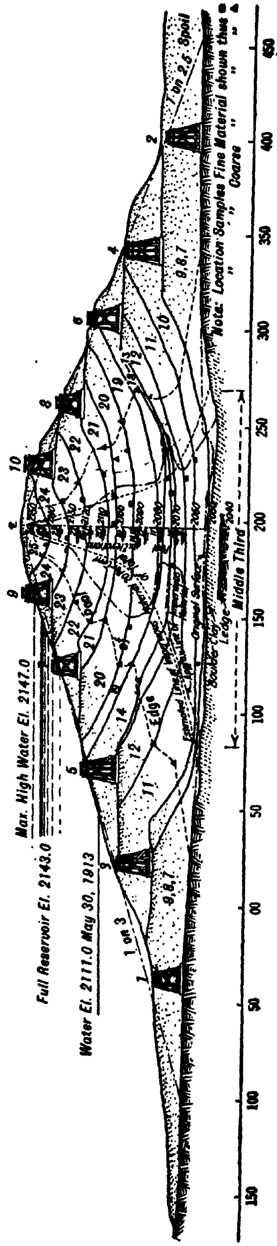

FIG. 158.—Somerset Dam, Vermont, Built by Semi-hydraulic Fill Method.*

* From "Some Investigations and Studies in Hydraulic Fill Dam Construction," J. Albert Holmes, Trans. Am. Soc. C. E., Vol. LXXIV, p. 231.

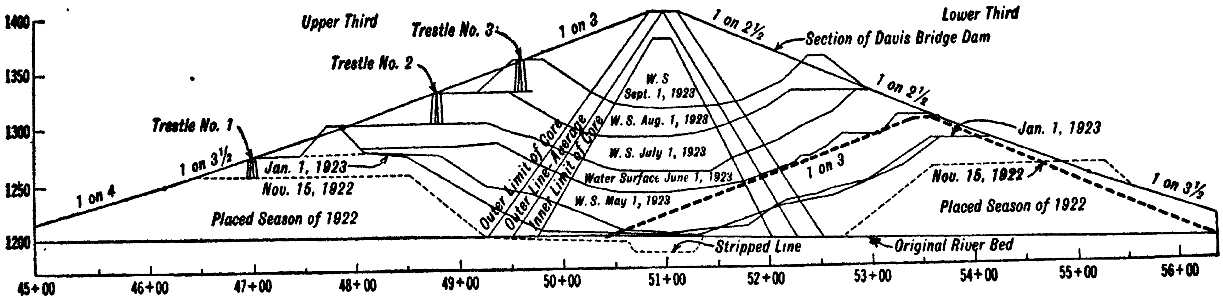

FIG. 159.—Davis Bridge Dam, Vermont, Showing Progress Cross-sections.* Built by Semi-hydraulic Fill Method.

* From Eng. News-Record, Vol. 92, p. 235.

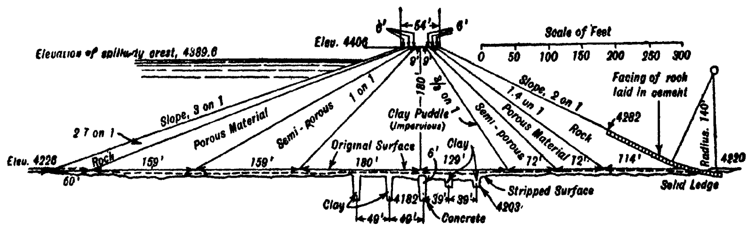

FIG. 160.—Cross-section of Dam No. 2 at Necaxa, Mexico, showing Dimensions, Cut-off Trenches and Theoretical Distribution of Materials.* Built by Semi-hydraulic Fill Method.

* From Allen Hazen in Trans. Am. Soc. C. E., Vol. LXXXIII, p. 1715.

whether the main drainage of the core is upwards or sideways, the importance of pervious outer sections is equally great.

In the case of dams built by the semi-hyrdaulic fill method, the toes and faces consist of car-dump fills, in most cases. Material is washed away from these fills by jets of water from giants. The finer material goes into a central pool and is deposited, forming the core, while the coarser particles are dropped near the car-dump fill. In consequence of this action, the car-dump fill at the face is generally more dense and impervious than the material immediately adjoining it on the inside of the dam, for this latter material has had the fines washed out of it by the action of the monitors.

Mechanical analysis of samples from the cross-sections of Paddy Creek and Linville dams makes it clear that the material in the car fills is actually more dense and impervious than that immediately adjoining. Through the presence of the central pool and the sluicing operations, this comparatively pervious area is kept full of water, which exerts considerable hydrostatic pressure on the relatively impervious car-fill material at the face. Thus the car fills form an element of weakness and, by imprisoning the water, may sometimes be the cause of slides even though the central core of fine material is kept very narrow.

This condition may be partly overcome by placing numerous drains through the car fills so that the more pervious material just inside can drain out, without producing hydrostatic pressure on the car fill.

122. Shrinkage of Embankments.—The amount of shrinkage depends on the character of the material used and the care taken in compacting it. Dams should be filled to a somewhat greater height and width than the neat dimensions finally desired. For an embankment rolled in 6 in. layers, as herein outlined, 2 per cent would be an ample allowance for shrinkage. With a sandy soil there is very little shrinkage, sometimes so little that it can scarcely be detected, if the material has been properly compacted. Cores of clayey material, deposited hydraulically, will settle much more than this, sometimes as much as 8 or 10 per cent. Material in a well-compacted earth dam, built in rolled layers, generally occupies considerably less space than it did in its natural state in the borrow-pit. For instance, cross-sections on the North Wing of the Olive Bridge Dam showed that this material, consisting mostly of " rock flour " with 8 per cent of small stones sprinkled through it, actually occupied 8 per cent less room in the embankment than in the borrow-pit.

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James D. Schuyler, Trans. Am. Soc. C. E., Vol. LVIII, p. 196.

SEVIER BRIDGE, UTAH,
Eng. News-Record, Vol. 76, p. 398.

JUNCTION DAM, MICH.,
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LITTLE HORSE CREEK DAM, S. C.,
Eng. Record, Vol. 60, p. 384.

THE CONCUNULLY DAM, IDAHO,
Eng. Record, Vol. 59, p. 368.

OCHOCO DAM, ORE.,
Eng. News-Record, Vol. 86, p. 291.

EARTH DAMS OF MIAMI CONSERVANCY DISTRICT, OHIO

Eng. News-Record, Vol. 85, p. 612.
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Recent Practice in Hydraulic-fill Dam Construction, by James D. Schuyler. Trans. Am. Soc. C. E., Vol. LVIII, p. 237.

CHAPTER XIV

ROCK-FILL DAMS

BY JOEL D. JUSTIN

124. General.—Rock-fill dams are used quite extensively in the West, in locations where the cost of transporting cement to the job would be extremely high. A number of these dams are over a hundred feet in height. They have been found especially applicable where the site consists of a narrow canyon with precipitous sides rising to a considerable height above the top of the dam. At such a site it is often possible to drive powder drifts into the walls of the canyon in such a way that a considerable portion of the rock fill required may be shot down to, or near, its final position in the dam. At the Morena rock-fill dam for the water supply of San Diego, Calif., 180,000 tons of rock was moved at a single blast. However, cableways or derricks are almost always necessary for rehandling the rock to its final position in the dam.

The possibilities of this type of structure are indicated by the fact that a rock-fill dam having a height of 780 ft. has recently been seriously proposed for construction in the Colorado River.[1] The Dix River Dam in Kentucky is 275 ft. high and contains 1,700,000 cu. yd. of rock fill.[2]

125. Rock-fill Dam with Impervious Deck.—This type of rock-fill dam consists essentially of an impervious upstream deck supported by the rock fill. This deck is usually formed either by a continuous reinforced concrete slab or by timber planking. Timber planking should never be used above low-water surface unless a cheap, temporary structure is desired, and preferably should not be used above about 20 ft. depth of water.

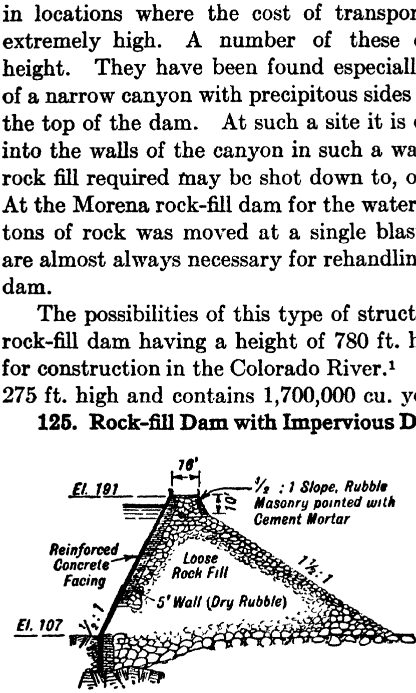

FIG. 161.—Section Through Beaver Park Dam, Colorado.
(Eng. News, Vol. 73, p. 660.)

The deck should rest on hand-placed or derrick-placed stone. Beyond this, the dam consists of a dumped fill of loose rock. It is not considered safe practice to make the entire dam of a dumped fill of loose rock, as the fill will settle, often causing the rupture of the deck.

[1] Tentative plan for rock-fill dam, Lee's Ferry, Ariz., E. C. La Rue, Trans. Am. Soc. C. E., Vol. LXXXIV (1923), p. 200.
[2] Eng. News-Record, Vol. 94, p. 648.

At the bottom, the upstream deck is carefully keyed into the ledge rock or to sheet piling extending to an impervious stratum. After the cut-off at the upstream toe has been constructed, the body of the dam is built. This consists of a loose rock fill having a downstream slope of 1 on 1 to 1 on 1½. This is accomplished by dumping the rock from derricks or cableways or by

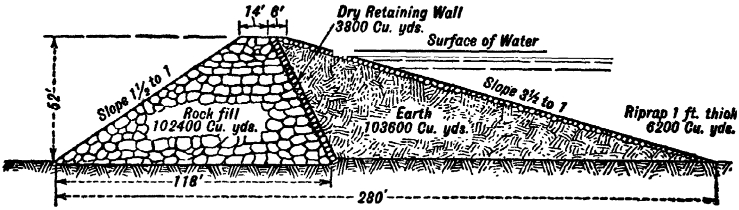

Fig. 162.—Pecos Valley Dam No. 2. New Mexico. (Wegmann).

direct blasting from the canyon walls. Earth or fine débris should be excluded as far as possible from this loose rock fill as, if it is included, it will eventually be washed down, causing undue settlement of the fill.

On the upstream side of the loose rock fill, it is good practice to have a section of large stone carefully placed in position by derricks, as indicated by

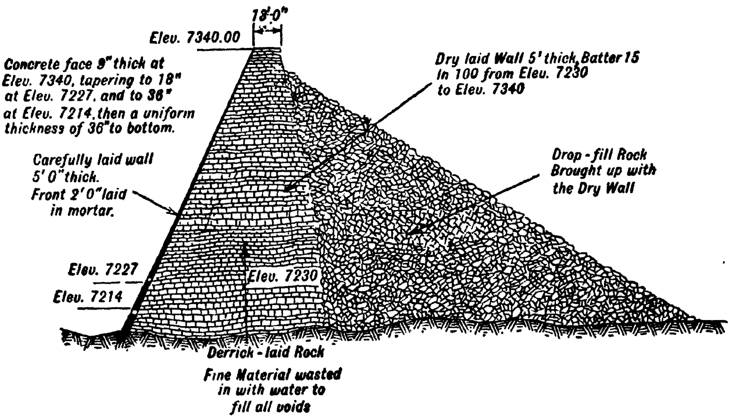

Fig. 163.—Relief Dam on Stanislaus River, California. (Trans. Am. Soc. C. E., Vol. LXXV (1912), p. 55).

Fig. 165. The thickness of this section varies from 6 to 50 ft. Upstream from this section, there is sometimes included a wall of rubble masonry laid in cement mortar from 3 to 6 ft. thick, as was done at the Morena dam and also at the Chatsworth Park dam in California. This wall then serves as a support for the water-tight deck of reinforced concrete.

In most cases, however, the deck of reinforced concrete or planking rests

on a dry rubble wall, and sometimes the section of large derrick-placed stone is omitted and the dry rubble wall rests directly on the loose rock fill. The slope of the upstream deck varies from a little steeper than ½ on 1 to 1 on 1½.

126. Spillway Provisions.—When the depth of overflow would be very small, the rock-fill dam is sometimes used as a spillway. For this purpose the downstream face is sometimes paved with dry rubble or is covered with a concrete slab. The objection to this procedure is that the loose rock fill composing the downstream portion of the dam is practically sure to settle, leaving the facing of concrete or rubble more or less unsupported. In some cases the spillway requirements are met by constructing a spillway over the rock fill out of timber and planking, as was done in the case of the Beaver Park dam in southern Colorado. Round timbers are generally used for sleepers, and a timber flume of heavy planking is constructed down the downstream face of the rock-fill dam. Such a timber-flume spillway is quite flexible and will

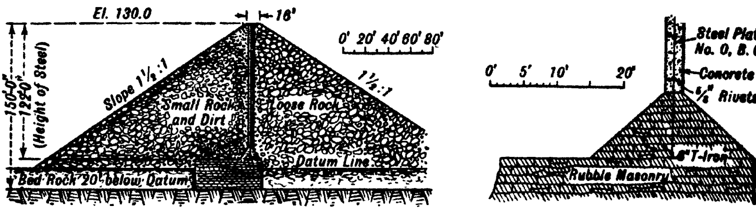

FIG. 164.—Cross-section of Lower Otay Dam and Enlarged Section of Steel Core Wall at Base.

Reproduced from "Engineering News," Mar. 10, 1898; dimensions of dam as built vary from this. (Eng. News. Vol. 75, p. 334.)

generally take the distortion due to settlement of the fill without serious rupture.

It is entirely possible to build a rock-fill dam for almost any condition of overflow, provided that all of the conditions are appreciated and given proper weight in the design. But to obtain a safe structure of considerable height for a material depth of overflow would frequently require so expensive a design that it is more economical to secure the required spillway capacity in some other way. The Laguna weir in the Colorado River, built by the United States Reclamation Service, is essentially a rock-fill overflow dam; but its height is 24 ft. and it has a bottom width of over 200 ft. and a downstream slope of 1 on 12.

Wherever it is at all feasible, the spillway for a rock-fill dam is provided for by means of a ledge-rock spillway channel or else through a tunnel in the wall of the canyon.

127. Core-wall Type of Rock-fill Dam.—Rock-fill dams are sometimes built with a core-wall of concrete or reinforced concrete. A steel diaphragm, with a thin protecting layer of concrete on each side, has also been used as a core-wall in such a dam. The core-wall is usually placed about in the center of the dam and the loose rock fill dumped on each side of it. The Lower Otay

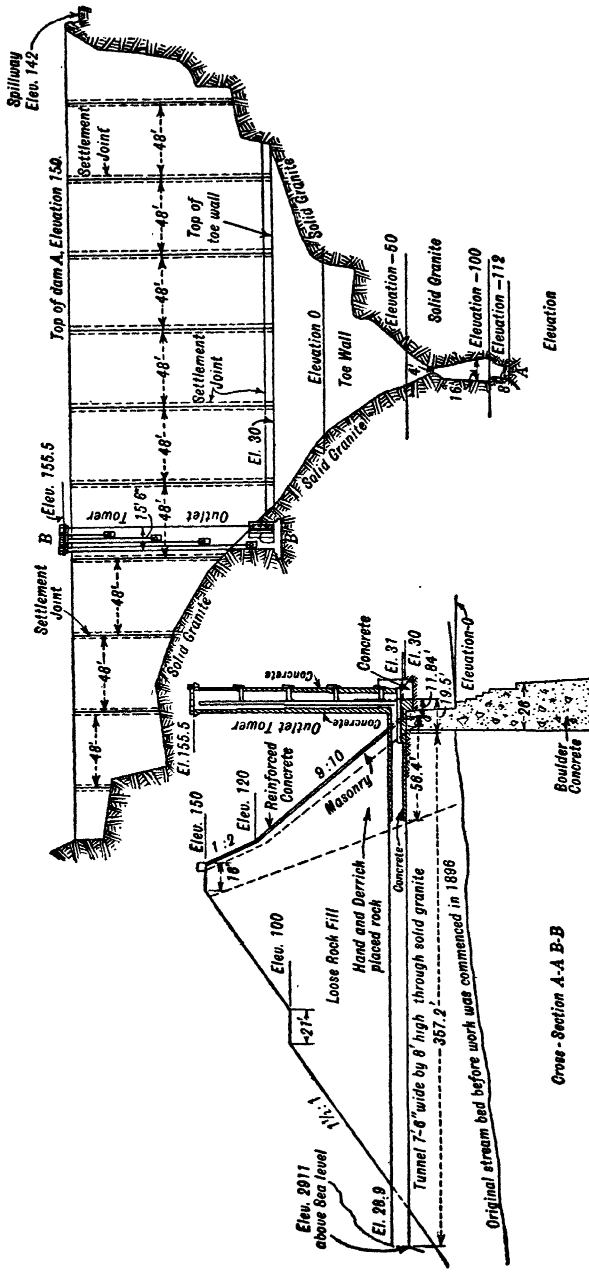

FIG. 165.—Morena Rock Fill Dam, California.
(Trans. Am. Soc. C. E., Vol. LXXV (1912) p. 37).

dam, in California, was of this type. This dam was built in 1897 to a height of 135 ft. and, after nearly twenty years of service, failed by overtopping in 1916. A dam of this type is practically an earth dam of excessively pervious material, depending absolutely on the imperviousness of the core-wall. In

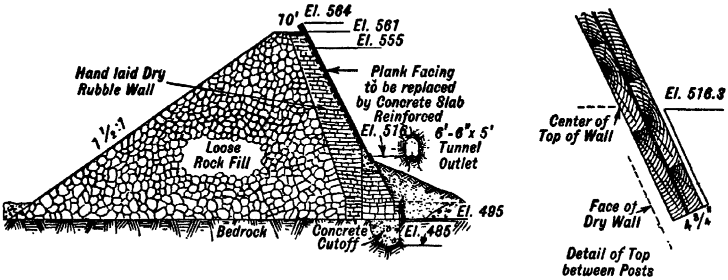

FIG. 166.—Section through Drew's Dam, Goose Lake Valley Irrigation District in Oregon. (Eng. News, Vol. 77, p. 100).

turn, the stability and safety of the core-wall depends on the supporting power of the loose rock fill. It is essential that there should not be much earth or débris in the rock fills, as this leads to undue settlement which may unbalance the pressures on the core-wall to such an extent as to cause serious cracks and distortion, and, in an extreme case, bring about failure.

128. Composite Type of Rock-fill Dam.—This type consists of a rock fill

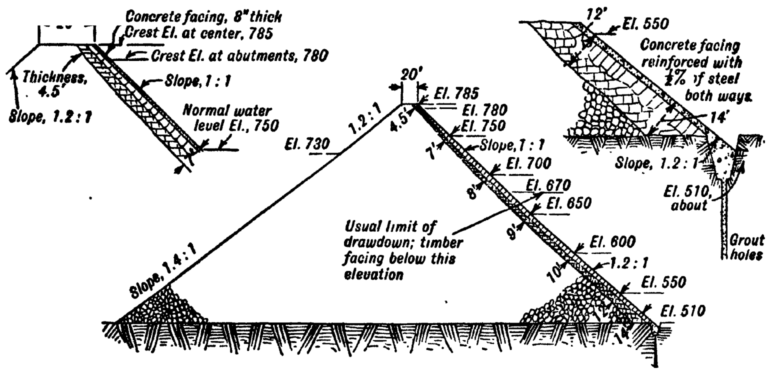

FIG. 167.—High Rock-fill Dam over Dix River—Rubble, Concrete and Timber Facing on Upstream Side. (Eng. News-Record, Vol. 94, p. 1059).

on the downstream side of the dam and an earth fill on the upstream side. Such a dam, when properly constructed, produces a very stable and satisfactory structure. The earth fill furnishes the water-tight portion of the dam, and the rock fill forming the downstream portion provides ready drainage

for seepage water and adds a greater degree of stability to the structure than would be provided, in most cases, by an equivalent amount of earth.

The Pecos Valley dams Nos. 1 and 2 are of this type. The Clear Lake dam of the Klamath project and the Snake river dam of the Minidoka project, both built by the Reclamation Service, are also of this type. In constructing such dams, the rock fill should be carried up ahead of the earth. Where the earth-fill and the rock-fill portions of the dam join, the voids between the big stones should be chinked up with smaller stones and then a heavy layer of crushed stone or gravel should be placed on the upstream surface of the rock fill before the earth fill is placed against it. In effect this measure produces a filter and prevents the earth from being carried away through the large interstices of the rock fill by the action of seepage water and rain.

129. Settlement.—Rock-fill dams are subject to considerable settlement. The amount of this settlement depends on the care with which the work is done, the character of the materials used, and, very largely, on the amount of earth and débris that is deposited in the rock fill along with the rock. In ordinary good work the settlement should not exceed from 3 to 5 per cent, and it is possible to insure a smaller percentage of settlement than this. On the other hand, if the entire structure is a loose rock fill in which a considerable percentage of earth and débris is included, the settlement may be very great indeed.

130. Bibliography.—

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CHAPTER XV

HEAD-WATER CONTROL AND ACCESSORIES FOR DAMS

BY WILLIAM P. CREAGER

131. Head-water Control.—All dams are provided with spillways to discharge the excess flow not used by the turbines. A spillway of the simplest type consists of an unobstructed overflow, the water rising and spilling over the crest as the flow increases. Very frequently, the elevation to which head-water may be allowed to rise is limited by land or water rights which are not owned. Therefore, with a simple overflow spillway, the crest of the dam must be limited in elevation to provide sufficient margin for the rise of water surface during floods. This, of course, requires a sacrifice of head which otherwise could be used for the generation of power. For this reason, the simple spillway should be as long as possible, consistent with economy, to limit the head on the crest during floods. If sufficient length of simple crest cannot be obtained, special devices may be used which, automatically or by manipulation, can be made to vary the discharge capacity of the crest and limit the fluctuations of the elevation of the pool to very narrow margins.

The usual devices used for this purpose may be divided into the following classes:

(1) *Crest Control,* which lowers or raises the crest as the river discharge varies;
(2) *Crest Gates,* which are opened and closed to vary the capacity of the spillway as desired;
(3) *Sluice Gates,* which are placed in the lower part of the dam to assist the spillway in the discharge of the flood; and
(4) *Siphon Spillways,* which increase the capacity of the spillway by providing a suction head in addition to the head on the crest.

The foregoing classes may be further divided into the following usual forms or types:

(1) Crest Control.
- (1*a*) Temporary flash-boards,
- (1*b*) Permanent flash-boards,
- (1*c*) Drum crests,
- (1*d*) Tilting gates,
- (1*e*) Bear-trap crests.

(2) Crest Gates.
 (2*a*) Sliding gates,
 (2*b*) Roller gates,
 (2*c*) Taintor gates,
 (2*d*) Rolling gates,
 (2*e*) Stop-logs and needles.
(3) Sluice Gates.
 (3*a*) Sliding and roller gates,
 (3*b*) Butterfly valves,
 (3*c*) Needle valves,
 (3*d*) Cylinder gates.
(4) Siphon spillways.

Not only is it impossible to describe within a limited space all of the many devices used for the control of head-water, but it is very difficult to define all of the contingencies which affect the choice of type. For cases where water is very valuable, it is essential to adopt a type that is tight and will not waste the flow during periods of drought. For rivers containing much ice or a large amount of débris during floods, the crest must be unobstructed or provided with piers that are sufficiently far apart to prevent stoppage.

The necessity for confining the fluctuations of head-water within very narrow limits exists only when the land for flowage or water rights is absolutely limited or very expensive and the head for power is sufficiently valuable to permit the expenditure for efficient head-water control devices. Such devices are very expensive for control within narrow limits, if the flood discharges are great. Obviously, the need for close control is greater in low-head developments than in high-head plants because, for the latter, a few feet additional head increases the ouptut a relatively small amount. Therefore, devices that only partly control the fluctuations are frequently used, since these are relatively less expensive.

Some references to various types of head-water control devices are given in Sec. 148. Essential information regarding the various types of gates, valves, and hoists required for head-water control for dams is contained in Chapter XVI, " Conduit Intakes," Sections 154 to 170 inclusive.

132. Temporary Flash-boards.—Flash-boards consist of a series of vertical boards or panels placed on the crest of the dam for the purpose of raising the pond level. Fig. 168 shows a typical system of temporary flash-boards, consisting of a series of panels supported by pins or pipes which are inserted loosely into sockets set in the masonry crest of the dam. The pins or pipes are designed to bend over and loosen the flash-boards when the water surface in the pond reaches a certain elevation, thus automatically lowering the crest to pass excessive floods. In this country, this is the most common of all devices designed to control the elevation of flood-water surface. The boards or panels are fastened loosely to the supports and are lost when the supports bend over, unless removed before anticipated high water.

In order to facilitate the handling of a barge for removing and restoring the flash-boards, it may sometimes be found desirable to provide, at intervals,

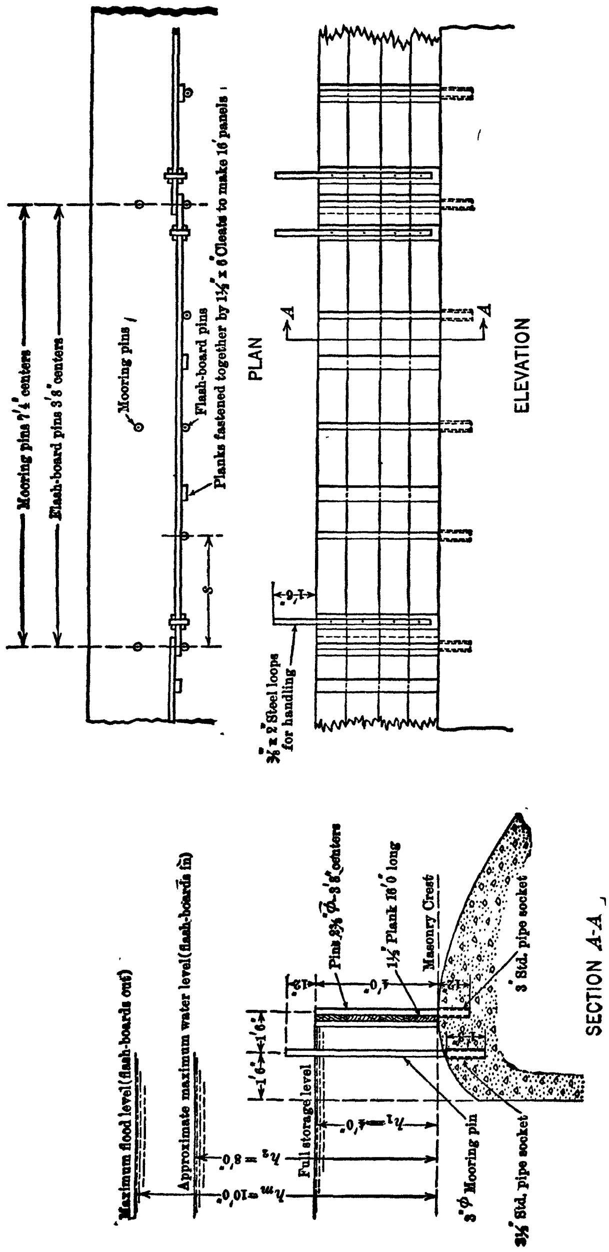


Fig. 168.—A Typical Temporary Flash-board Installation.

sockets into which mooring pins may be set, as indicated in Fig. 168. Cableways may also be used as an anchorage for the barge. The boards and supports are sometimes manipulated from an overhead trolley or bridge on the crest of the dam.

Where it is possible to remove the flash-boards in advance of floods, they are usually built in panels, as indicated, and provided with handles. For the type shown, handles are not permissible if considerable drift is anticipated, as this may collect and cause premature failure. The boards usually have unplaned edges, and ashes or similar calking materials are used to make them tight.

It is desired that, when the pins fail, they will bend over quickly and lie flat on the dam to leave an unobstructed crest. Plain, solid pins do not fully meet these requirements. For this reason, circumferential notches have occasionally been provided in solid pins at crest level, to insure accurate diameter and to make the failure sudden and complete. The writer has found that plain steel pipe makes the most satisfactory pin. It will not fail by buckling or shear unless stressed to failure in tension. After tensile failure starts, however, the pipe collapses at the crest level and the bending is quick and flat to the crest.

Tests on steel-pipe pins of various weights, sizes, and lengths indicate that failure will occur for a computed stress [1] of between 42,000 and 58,000 lb. per square inch. These tests are indicated in Fig. 169. The variations in the stress at failure are due to variations in the fabrication of the pipe and to variations in the vacuum, which is usually present under the sheet of spilling water and adds to the load on the pins. The variations in calculated stress at failure, as shown by the tests of Fig. 169, correspond to relatively small variations in water surface.

The diagram of Fig. 169 shows the relation between the various elements affecting the design of flash-board pins. Assume that it is desired to design flash-boards 3 ft. high (H) to fail when the water surface rises to a height (h) of 2 ft. above their top, or 5 ft. on the permanent crest. Let it be assumed that the pins will fail for a calculated stress (K) of 50,000 lb. per square inch, and that they are to be spaced 5 ft. centers (d).

To find the required size of pins, find $H = 3$ ft. at the left-hand margin, trace horizontally to intersect $h = 2$ ft., thence vertically to $K = 50{,}000$ lb., thence horizontally to intersect $d = 5$ ft., and thence vertically to the lower margin, and read a required section modulus of 1.05. The scales on the lower margin also show that a 2½-in. standard pipe or a 2¼-in. solid pin would answer.

In order to show the variation in water-surface elevation for different assumed stresses at failure, assume that the pins in the foregoing example will fail for a calculated stress somewhere between 42,000 and 58,000 lb. Proceeding in the reverse direction in the diagram and using a pin with section modulus of 1.05, we find that, for 42,000 lb., h is 1.5 ft. and, for 58 000 lb., h is 2.6 ft. Therefore we can say that the flash-boards, as designed, will fail with a depth of water over the top of somewhere between 1.5 and 2.6 ft. with a probable depth of 2.0 ft. A few failures will indicate more accurately the

[1] Straight-line stress formula.

depth at which, for the particular installation, the pins will bend over, and a change in size can then be made if considered desirable.

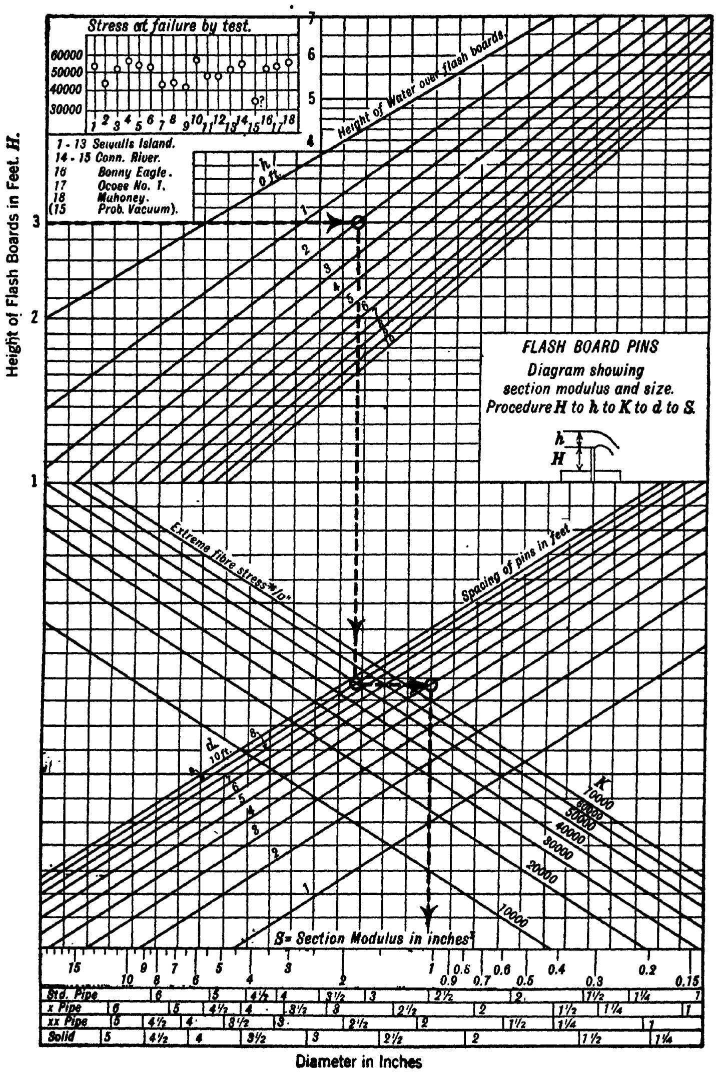


FIG. 169.

In some cases, the panels have been arranged as shown in Fig. 170. When the support, A, is removed, the panels revolve progressively around the pins

until all the panels are either swept away or left hanging to the pins as shown by the dotted lines. This scheme leaves the pins in place and they can be used over again. However, if the river is subjected to a considerable run of débris during floods, enough of this may accumulate on the pins, after the panels have gone, to obstruct the crest without bending the pins, thereby causing higher water during floods than has been anticipated.

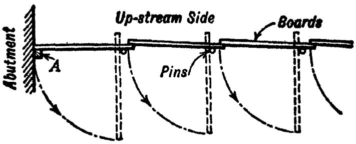


Fig. 170.—Plan of Flash-boards Designed for Progressive Failure.

Temporary flash-boards are always advantageous for any dam not otherwise equipped with crest control, provided that the pond does not lap the development next above. Pipe sockets should be provided in all crests of such installations, whether flash-boards are contemplated in the near future or not. Temporary flash-boards have been used up to a height of 10 ft.

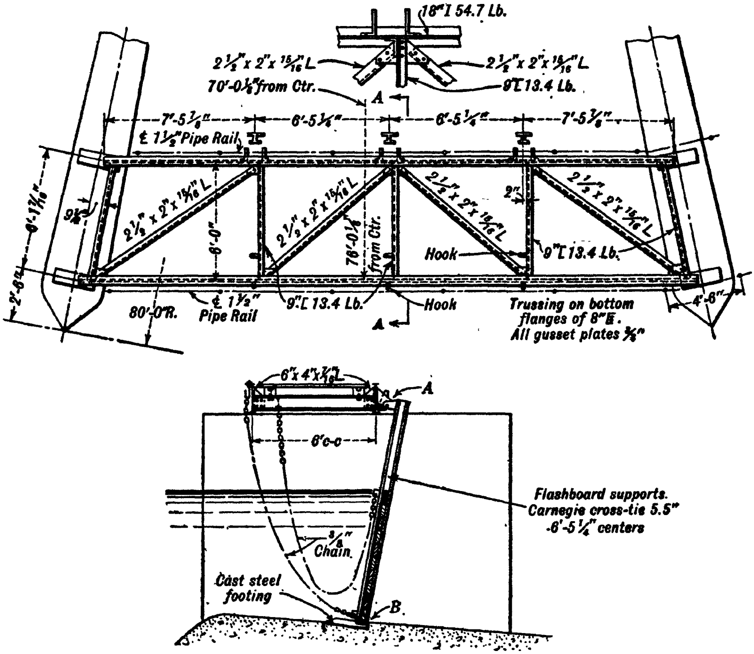


Fig. 171.—Details of Flash-board Device, Davis Bridge Spillway.
(E. A. Dow at spring meeting, Am. Soc. Mech. Engrs., 1925.)

After the boards are removed by a flood, the operators are usually able to replace them before the flow is reduced to the capacity of the turbines and the pond refilled by the tail end of the flood. With sufficient capacity of sluice

gates or other emergency outlets, the water surface can be lowered to the elevation of the permanent crest before the flood has entirely receded, and the flash-boards can be easily replaced. The sluice gates are then closed and the pond filled to the top of the boards.

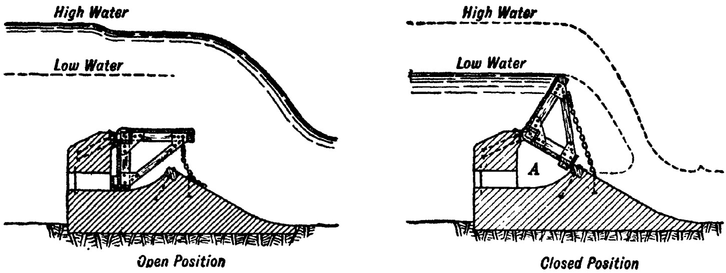


FIG. 172.—Stickney Type of Drum Gate.

Interesting experiences with flash-boards at the McCall Ferry Dam are related in Reference 13, Sec. 148.

133. Permanent Flash-boards.—Permanent flash-boards are similar in principle to the temporary type, except that they are designed to operate, automatically or by manipulation, without damage to themselves. They have been used mostly for special conditions and have not come into general use.

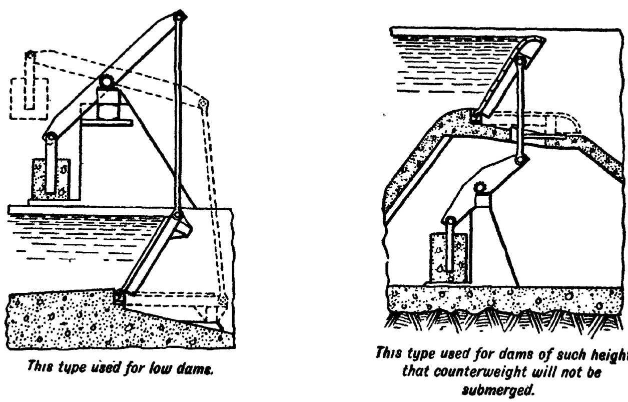


FIG. 173.—The Stauwerke Tilting Gage. Fargo Eng. Co., Jackson, Mich., Agents.

Figure 171 shows details of the permanent flash-boards used at the Davis Bridge Dam of the New England Power Company. The supports are held in place by a seat on the crest at *B* and a latch on the bridge at *A*. They are removed during floods by tripping the latch. The boards, which are in the form of stop-logs, are lost, and the supports are drawn up to the bridge by means of the chains and returned to place after the flood by fastening the top

to the latch and swinging them down into the seat at the bottom. The new boards are then forced down from the top. The piers of Fig. 171 are not parallel because the Davis Bridge spillway is curved.

One reason for using this type of flash-board, in preference to the less expensive temporary type previously described, is that the Davis Bridge spillway controls an extremely large reservoir. The tail end of the flood would not be of sufficient duration to refill such a large reservoir if the water surface were allowed to drop to an elevation close to that of the permanent crest in order to replace the flash-boards, there being no other outlet of large capacity to be used to discharge the tail end of the flood while the boards are being replaced.

Many other types of permanent flash-boards have been used. Several are mentioned in Sec. 148.

134. Drum Gates.—Fig. 172 shows the Stickney[2] type of drum gate used for the New York State Barge Canals. Constant head-water pressure is admitted to the compartment A, and this pressure on the lower leaf of the gate holds it, during low water, in its upper or closed position. During floods, the greater force of water pressure on the upper leaf, which is the longer leaf, forces the gate to its lower or open position. When the flood passes, and the water surface drops to the low-water level, the gate again rises.

Other forms of this type of gate have been used. See also Items 2 and 17 of Sec. 148.

135. Tilting Gates.—Tilting gates of many types have been installed, most of which have been patented devices. Two types are shown in the accompanying illustrations. The Stauwerke gate, shown in Fig. 173, and the

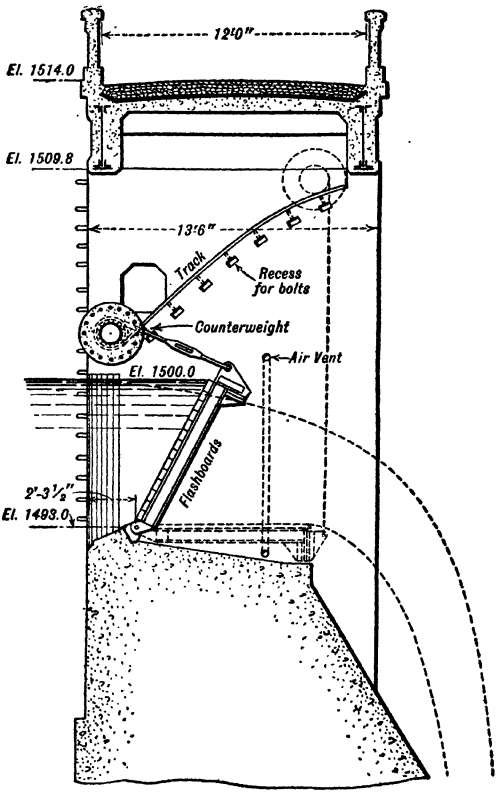


FIG. 174.—Tilting Gate for Tallulah Falls Development. Eng. Record; Vol. 69, p. 326 (March 21, 1914).

[2] A patented device. G. F. Stickney, Albany, N. Y., patentee.

Tallulah Falls gate, shown in Fig. 174, are designed to tilt as the water rises. The degree of regulation of water surface is such that only several inches' rise will cause the gates to tilt. The gates return to their closed position when the water surface again becomes normal. The gate of Fig. 173 is balanced by a counterweight, as shown, and that of Fig. 174 by a rolling counterweight which travels on an inclined track and rolls up the track as the gate tilts.

136. Bear-trap Dams.—Fig. 175 shows a sketch of a typical bear-trap dam. Dams of this type are used quite frequently for log sluices, but not extensively for crest control. The dam is raised by admitting head-water pressure to the under compartment, A, the pressure being sufficient, if the gate is properly proportioned, to overcome both the weight of the gate and such water as may be passing over it. Excluding head-water from compartment A, and draining it to the tail race, lowers the dam.

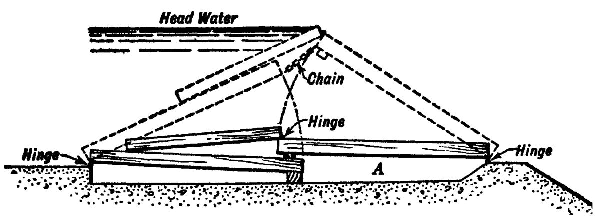


Fig. 175.—Typical Beartrap Dam.

Bear-trap dams are not very tight. An example of installation is mentioned in Item 8 of Sec. 148.

137. Crest Gates.—Crest gates are supported between piers placed at regular intervals along the crest of the dam and are operated from a hoist on an overhead bridge. The gates should be capable of lifting clear of the projecting branches of floating trees, and are frequently protected by a concrete buffer or front wall above water surface, as shown in Fig. 177. The several types of crest gates usually employed are described in succeeding sections.

For very large installations, as at the Wilson dam (Fig. 176) and at the Gatun Spillway of the Panama Canal, large gates of the Stoney type are usually employed. The caterpillar gate is also adaptable at such places.

Sliding lift gates are seldom used on account of the large capacity of hoist required to overcome the excessive friction. It is usually found more economical to use fewer piers and adopt a large-sized gate of a type easily lifted.

Taintor gates (Fig. 177) are perhaps the most common type adopted for dams controlling medium-sized drainage areas, where floods are not excessive.

The rolling gate (Fig. 178) has been used to some extent abroad but has not come into general use in this country.

Stop-logs and needles are the least expensive, but are seldom used to a considerable height for flood control, on account of the difficulty in installing and removing them. Their use has been confined principally to the control of log- and trash-chutes.

Where only a few gates are installed, individual hoists are usually provided; but, when there are a large number of gates, traveling hoists are commonly used. A spare hoist should always be provided, in case traveling hoists are used, and several active hoists may be necessary if the gates must be raised quickly during floods. Hoists for crest gates are usually electrically operated as hand operation for large gates is very slow.

138. Stoney and Caterpillar Gates.—Fig. 176 shows a typical installation of

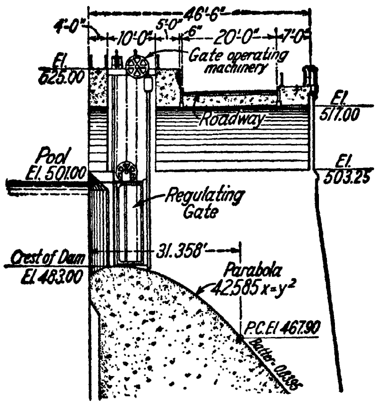


Fig. 176.—Stoney Crest Gate Installation at Wilson Dam, Alabama.

Stoney gates. Caterpillar gates are equally adaptable to this type of setting. Both are more fully described in Sec. 156, of Chapter XVI, " Conduit Intakes," and weights of Stoney gates are given in Fig. 210A. For very large installations of this type, a counterweight, located in a shaft inside the piers and extending into the dam, is frequently employed to lessen the duty of the hoists.

The gates of the Gatun Dam are 20 ft. high and 45 ft. long.

139. Taintor Gates.—Fig. 177 shows a common form of Taintor-gate installation with wooden face. A plate-steel face is more durable and is used more frequently. However, the wooden face has insulating properties which reduce the extent of freezing, as explained in Sec. 147. A wooden face, if used, should be creosoted. Counterweights are sometimes used with Taintor gates, of the type shown in Fig. 176 for a Stoney gate; but only for extremely large sizes. Taintor gates are more fully described in Sec. 157 of Chapter XVI, " Conduit Intakes." Items 6 and 7 of Sec. 148 refer to discriptions of other installations of this type. The usual maximum size of Taintor gates is about 20 ft. by 30 ft., but larger sizes have been installed.

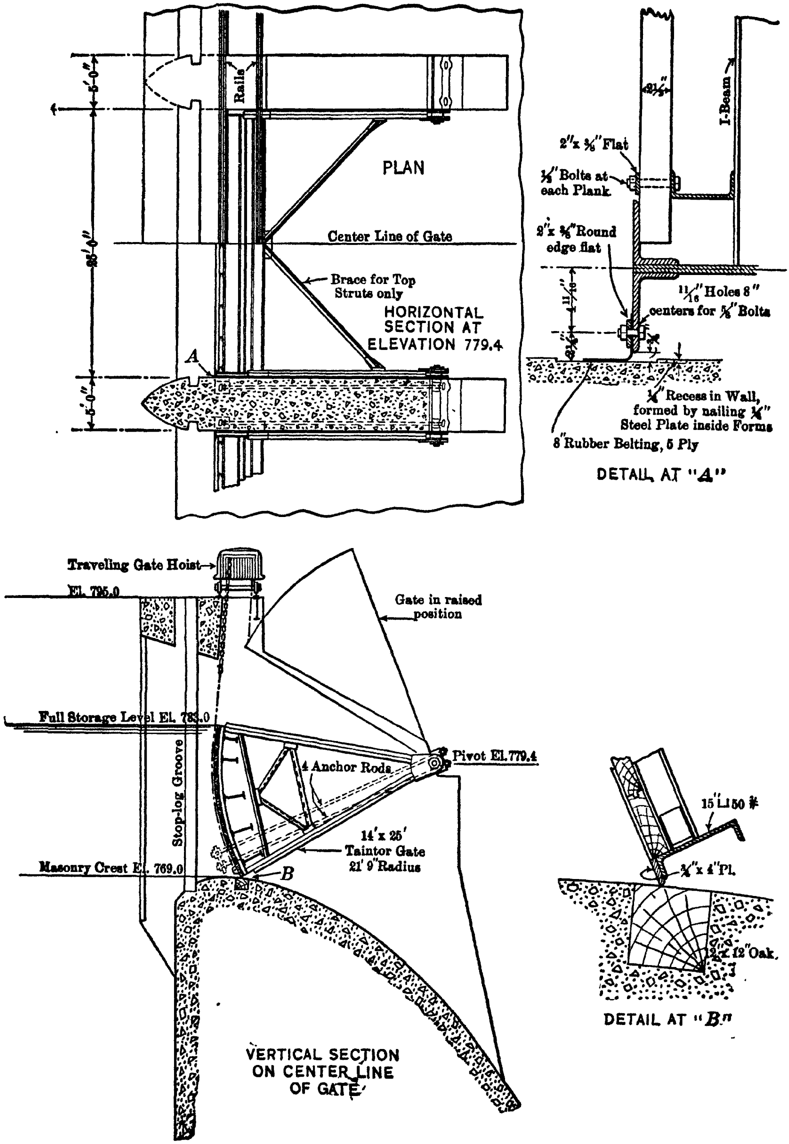


Fig. 177.—Taintor Crest Gate Installation at Great Falls Dam, Caney Fork River, Tennessee.

140. Rolling Gates.—The rolling gate illustrated in Fig. 178 has at each end a circular geared bearing which rolls on a geared track laid in a recess in the pier. The gate is lifted by a cable encircling the gate near the bearing.

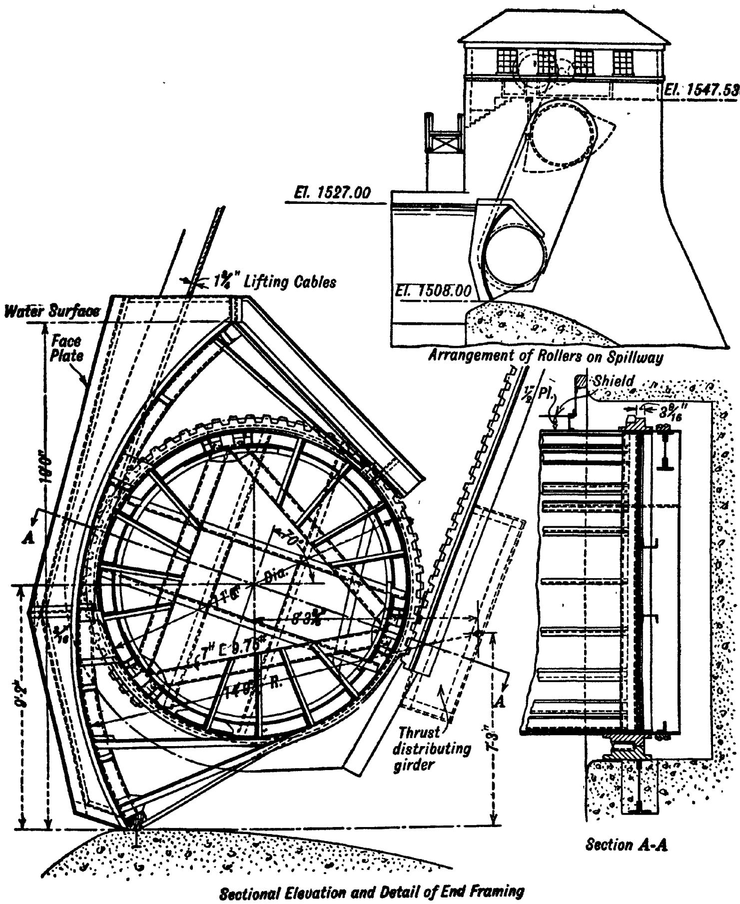


FIG. 178.—Rolling Gate at Long Lake Dam.
(Eng. Record, Vol. 70, p. 322).

Items 9, 10, and 11 of Sec. 148 refer to descriptions of other installations of this type. Such gates have been installed as large as 100 ft. long and 20 ft. deep.

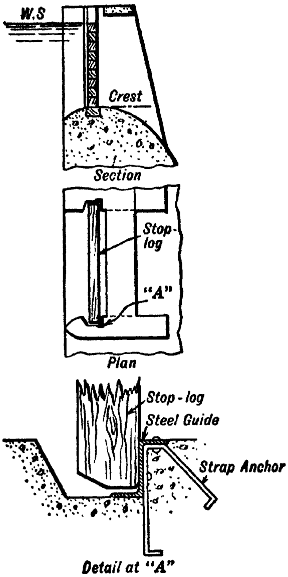


FIG. 179.—Typical Stop-log Layout.

141. Stop-logs.—Stop-logs are vertical layers of loose timbers spanning an opening between piers or abutments and supported at each end in grooves. A typical stop-log layout is shown in Fig. 179. Stop-logs are the simplest form of crest gates and are removed one by one as the need for increased discharge occurs. The chief objection to their use is the difficulty of installing and removing them.

Figure 180 shows a winch designed for their removal during periods when water is passing over them. This winch is further described in Item 5 of Sec. 148. Stop-logs are installed by pushing them down one by one with poles, and they are held in place by the water pressure.

Figure 223 shows a detail at the ends of stop-logs, which permits them to be reached by a hook for removal. This arrangement can be used if the water is not too deep or swift.

Stop-logs are chiefly used for the outlets

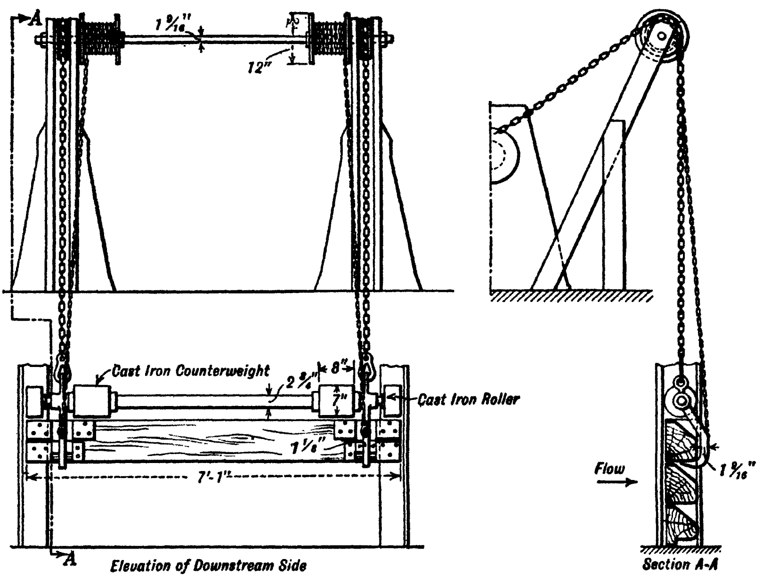


FIG. 180.—Weighted Hooks for Grappling Stop-logs.
(L. Ross in Eng. Record, Vol. 74, p. 114).

of small log- and trash-chutes and are usually removed by hand. They are extensively used for emergency purposes, to unwater all kinds of water-control apparatus.

The facility with which stop-logs can be installed depends greatly upon the smoothness of the guides. Therefore, wooden or steel guides are used for long spans and deep sluices, unless special provision is made for placing very smooth concrete for the logs to rest on. While leakage below low-water surface will preserve wooden guides, they will rot at and above water. Therefore, steel guides, similar to that illustrated in Fig. 179, are recommended.

142. Needles.—Needles consist of a row of slightly inclined timbers, supported at the top by a bridge or beam, and at the bottom by a sill on the crest, as shown on Fig. 181. The needles are placed one by one, by extending them horizontally over the pond and allowing them to tip into the current until the lower end swings down in contact with the concrete near the sill. They are then drawn slightly upward until the lower end rests on the sill, and are rolled sidewise into place against those needles already installed. They are removed by lifting each from its seat and hauling it out bodily. Each should be provided with a hole near the top through which to pass an anchor rope. The purpose of this rope is to hold the needle in case it gets away from the operators when being handled. Needles are invariably installed and removed by hand, and should not be too large. Timbers 6 in. by 6 in. and 20 ft. long have been used.

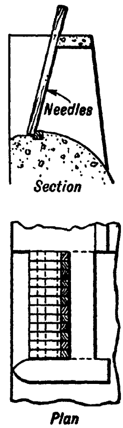


FIG. 181.—Typical Needle Layout.

This type of control is not frequently used.

143. Sluice Gates and Valves.—Sluices are frequently installed through concrete dams, to be used as outlets for storage reservoirs, to empty small ponds for repairs, to assist in the passing of floods, to facilitate the installation of flash-boards as described in Sec. 132, and for other similar purposes. Sluice gates and valves, or sluice "control," have been used in a variety of forms, including ordinary valves, rectangular sluice gates having both sliding and roller bearings, butterfly valves, needle valves, and cylinder gates. All of these are described in considerable detail in Chapter XVI "Conduit Intakes," the details of installation for conduit intakes being similar to those required for sluice control.

Any sudden change of section in a conduit will cause a partial vacuum accompanied by violent eddies. The combination of a vacuum and eddies is very conducive to erosion. Therefore, the entrance to the sluice should be shaped as nearly as possible like that of the standard orifice (Fig. 62) and the area of the sluice should be reduced gradually between the entrance and the control. A change of section will invariably be necessary at the control; consequently, if the control is not placed at the extreme lower end or outlet of the sluice, the concrete below the control must be well protected by a cast-iron or steel lining, as indicated in Fig. 182, well anchored, and provided with ample holes to prevent rupture due to the pressure of seepage from head-water.

It is now generally conceded that the sluice, from the control to the outlet, should be of the same section as the control. This is not the case in the sluices of which details are shown in the accompanying illustrations. The

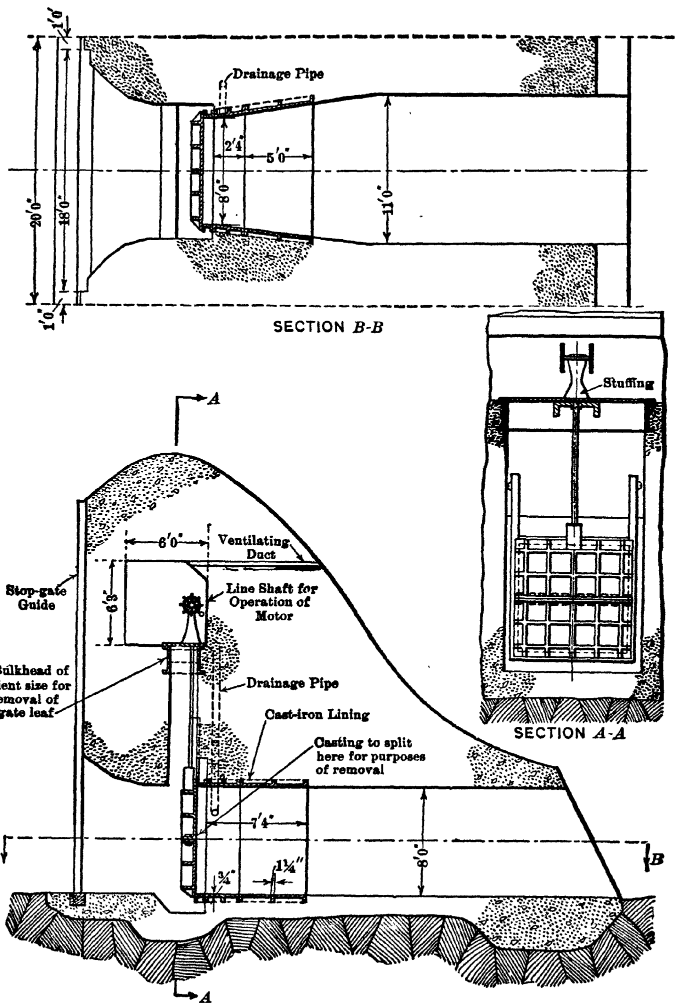


FIG. 182.—Arrangement of 8 × 8-ft. Sluice Gates, Stevens Creek Development on Savannah River.

enlargment of section shown in Fig. 182 is for the purpose of regaining some of the velocity head and increasing the discharge. However, a large percentage increase in discharge can be obtained, by this means, only for very low

heads, and the practice is objectionable on the ground that it increases the vacuum and hence increases the erosion.

Vents, admitting air to the sluice below the control, have reduced the vacuum to a considerable extent and should always be provided. However, it is not possible to install enough vents at the control to destroy entirely the tendency to a vacuum.

If the tail-water below the dam has considerable depth, the maximum capacity of the sluice is obtained when it is placed just below tail-water surface. However, in such cases, the sluice control is not accessible for repairs. If the sluice is placed above tail-water, some head is sacrificed and leakage water will freeze in cold climates. If the tail-water fluctuates sufficiently, the sluice may be placed below the water surface corresponding to the river discharge that occurs when the sluice is ordinarily operated, and will then be above water surface most of the time. Trouble from freezing can be elim-

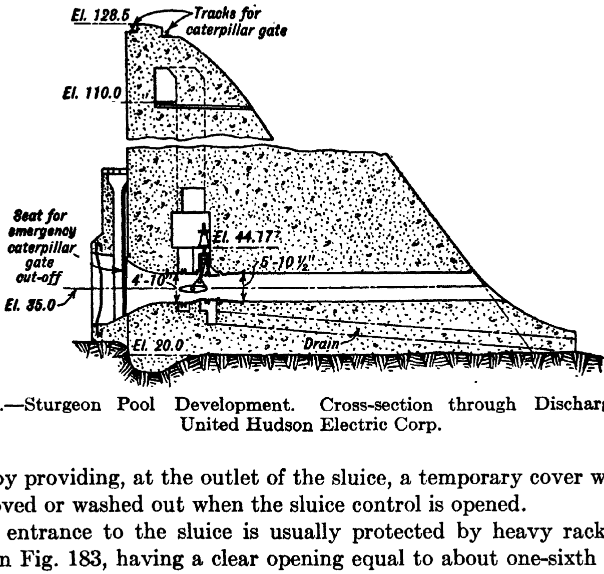


FIG. 183.—Sturgeon Pool Development. Cross-section through Discharge Valves, United Hudson Electric Corp.

inated by providing, at the outlet of the sluice, a temporary cover which may be removed or washed out when the sluice control is opened.

The entrance to the sluice is usually protected by heavy rack bars, as shown in Fig. 183, having a clear opening equal to about one-sixth the small diameter of the sluice control. However, some of the logs and trees which are stopped by the racks will pass part way through them, and may interfere with the closure of the sluice if the racks are too close to the control. Therefore the racks should be located 20 to 30 ft. above the control, depending upon the length of expected logs and trees. Racks are sometimes omitted, particularly for very deep pools.

Provision should be made for closing the upper end of the sluice to facilitate removal of the control for repairs. Stop-logs are used for this purpose in low dams. For high dams, a seat is provided to receive a bulkhead or gate, which is lowered from the crest.

The control is usually placed inside the dam, as shown in Fig. 183; but,

for high-head sluices, it is advantageous, as explained before, to place the control at the outlet, to provide free discharge and obviate danger from erosion. The latter location obviously cannot be used readily for sluices in spillway dams.

As the inside of the dam is very damp, geared hoists, especially if motor-controlled, require constant care to keep them in good condition. Oil-pressure cylinder hoists are most adaptable to sluice gates and valves inside the dam, as far as cost of maintenance is concerned.

All sliding surfaces of sluice controls, including the stems, should be fully and heavily bronze-mounted, as they are kept closed for long periods and are subject to heavy duty.

The tendency to form a vacuum is very pronounced when a sluice gate or valve is only partly open. The violent eddying below the disk, with consequent impact on the downstream face, causes periodical reversal of resultant

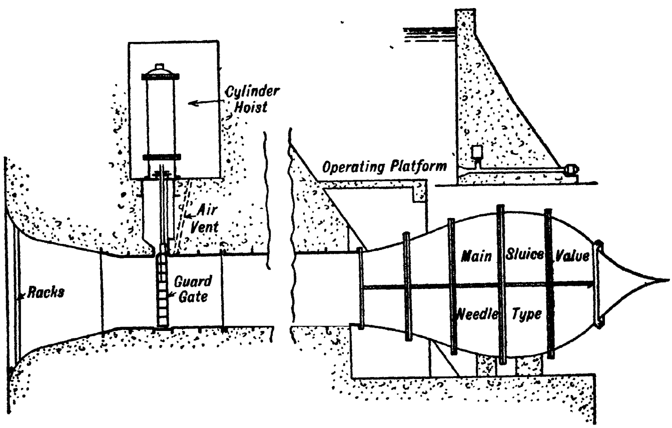


FIG. 184.—High-pressure Needle Valve Outlet.

pressure. Hence, a vibration, or "chattering," will occur if the disk is loose in its guides. For this reason sluice gates and valves are seldom used above about 50 ft. head as they are loose in their guides except when full open and shut. Butterfly and needle valves, on the other hand, are adaptable to the higher heads as they are free from chattering, having practically no play at any position.

Rectangular sluice gates, as indicated in Fig. 182, are most commonly used for low heads. Ordinary round valves are seldom installed, on account of their greater susceptibility to damage from chattering.

Because of the tendency to chatter, slide gates and valves should be made exceptionally rugged. Too much stress cannot be placed on this necessity. Some years ago the writer installed a manufacturer's standard sluice gate in a dam. It failed and was replaced by a much heavier gate. This also failed, and a third, still stronger, gate was necessary.

Roller-bearing gates have not often been used for sluices in the dam, as they are most adaptable to larger-sized openings than are commonly adopted for sluices. Moreover, they are operated to best advantage by a drum hoist, the multiple cables of which cannot readily be passed through the necessary stuffing boxes between the gate and the hoist.

Figure 184 shows a typical installation of a high-pressure dam outlet. The guard gates are frequently used in cases where it would be very difficult, on account of the great depth of water, to plug the upper end of the sluice for repairs to the control or lining. The guard gate, being operated very infrequently, is not likely to be damaged. Both needle and butterfly valves have been used in many installations for this type of outlet.

For a very high dam, it is advisable to install outlets at several different elevations so that they may be operated successively at relatively low head as the water is drawn down in the pond.

The best position for the valve is at the downstream end of the sluice; but it may be desirable to locate it in the middle of the sluice as shown in several other illustrations. If the valve is at the downstream end of the sluice, full uplift of head-water should be considered as acting in the vicinity of the sluice, unless the latter is lined and provided with ample cut-offs.

The reader is referred to Items 18, 19, and particularly Item 20 of Sec. 148, for a discussion of high-pressure outlets.

144. Siphon Spillways.—The siphon spillway [3] is a device for discharging water over a dam; it utilizes the available head to produce a higher velocity of flow than would be attained at an overflow wier, thus increasing the discharge.

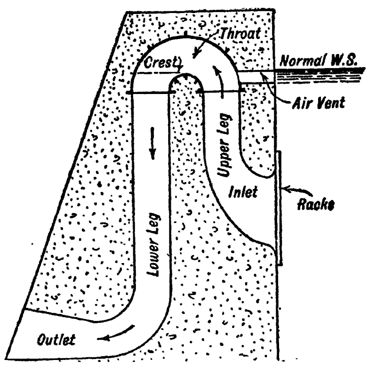


FIG. 185.—Simple Siphon Spillway.

In Fig. 185, the normal head-water surface is at the level of the crest of the siphon spillway. When the water rises, it spills over the crest; and, when the flow is such that the discharge strikes the downstream side of the lower leg, the air thus confined in the throat is quickly entrained and ejected, and the siphon primes. The suction thus produced increases the velocity to that corresponding to an effective head equal to the difference in elevation between head-water surface and the center line of the outlet, less the head expended in friction within the siphon.

If the full discharge of all the siphons at the dam is greater than the total flow into the pond, the head-water surface will fall. When the upper parts of the air inlets are exposed, the air drawn in by the suction reduces the efficiency of the siphons until the discharge is automatically diminished to that required for stationary water surface in the pond. If, however, two much air

[3] Patented by G. S. Stickney, Albany, N. Y.

is drawn in, the siphon action will be broken and the head-water surface will then rise again and the operation of priming will be repeated. If properly proportioned, the siphons will prime within a few seconds after the water has risen to the required elevation.

The upper leg is made of sufficient length to bring the inlet well below water surface, in order to prevent the entrance of ice and drift. The inlet is made two or three times the area of the throat and is well rounded and is usually protected by rack bars quite wide apart. The lower leg should be as long as is practicable, up to the siphon limit, to take advantage of all the head available. The outlet may be submerged or may be opened to the air, except for special cases where submergence is necessary.

The throat is frequently protected by a lining, to prevent erosion if the velocity is high. Provision should be made to by-pass ice cakes and large pieces of débris, which otherwise might clog the siphons.

Priming usually takes place when the water has risen a distance equal to about one-third the height of the throat and, for very large siphons, a throat height of 3 or 4 ft. has been used. The fluctuations of water surface may be

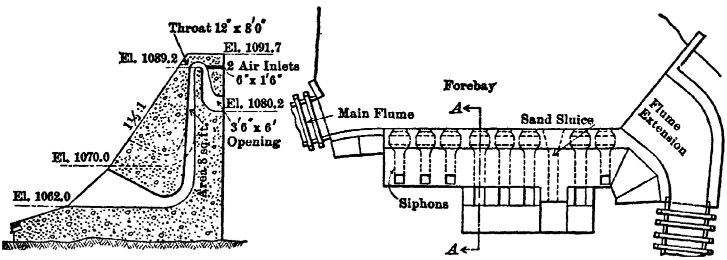


FIG. 186.—Siphon Spillway, Ocoee River Development, Tennessee.

Engineering Record, Vol. LXXII, p. 567.

confined within very narrow limits, even with such large siphons, by providing a small auxiliary priming siphon, having a small height of throat. Such a siphon will prime for a small rise of head-water surface, and is connected to the large siphons by a pipe which draws the air out of the large siphons and causes them to prime quickly. In such cases, the outlet of the large siphons must be sealed with water.

Figure 186 shows the siphon spillway of the Tennessee Power Company on the Ocoee River.

The discharge of a siphon of this type may be computed from the ordinary equation for flow through short tubes:

$$Q = CA\sqrt{2gh}, \quad \ldots \ldots \ldots \quad (114)$$

where h = the gross head on the siphons in feet from head-water surface to center line of outlet;

A = the area of the throat, in square feet;

g = the acceleration of gravity = 32.2;
Q = the discharge, in cubic feet per second; and
C = a coefficient depending on the characteristics of the siphon.

Tests by the writer on the Ocoee siphons indicated a value of C of about 0.65, and this figure has been fairly well substantiated by other tests on similar siphons.

145. Fish Ladders.—Fish ladders, to provide a means for the safe passage of fish past dams, are frequently required by State law. A fish ladder consists of an inclined trough in which the water flows from the upper to the lower pool at a velocity against which fish can easily swim. The three general types of fish ladders are shown diagrammatically in Fig. 187.

Sketch A of Fig. 187 shows the overflow type, in which each baffle board contains a V-shaped notch over which the water spills, with a fall low enough for the fish to jump. This type is not adaptable to all kinds of fish, as some sluggish species will not jump.

Sketch B is the rapids type, in which the baffle boards are shorter than the width of the trough, leaving openings at the end through which the water passes. The openings are staggered, as shown, so that the water takes a circuitous path, in order to kill as much of the velocity as possible.

Sketch C is the sluice type, in which each baffle board contains an opening through which the water passes. The openings are staggered as shown.

Specifications for fish ladders are furnished by the States that require them. The main object of the design is to provide as much friction as possible so that the slope of the ladder will be a minimum for a given velocity of flow. Changes in the direction of flow, forming frequent eddies, not only cause additional friction, but provide places for the fish to rest. The maximum velocity is

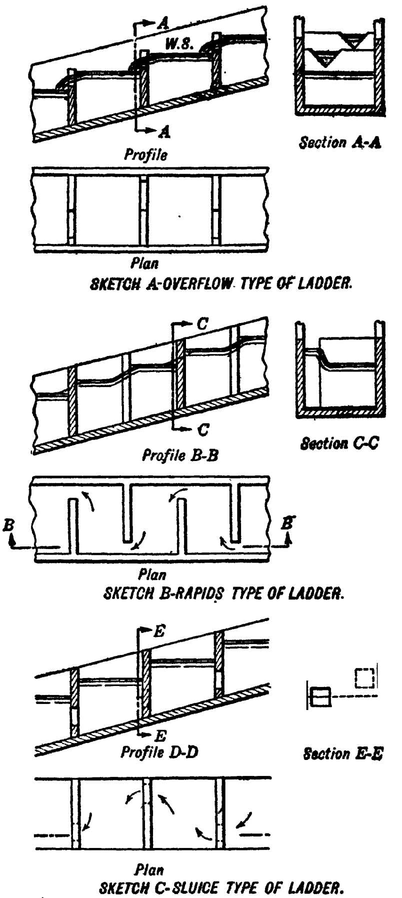


FIG. 187.—Diagrammatic Sketches of Types of Fish Ladders.

usually limited to about 10 ft. per second; but some species of fish can overcome much greater velocities.

The ladders may be protected by a grating but must not be entirely covered, as the fish will not pass a dark compartment. The habit of fish is to proceed up the river until halted by an obstruction, and then to nose their way from side to side, searching for a passage, always facing upstream. There-

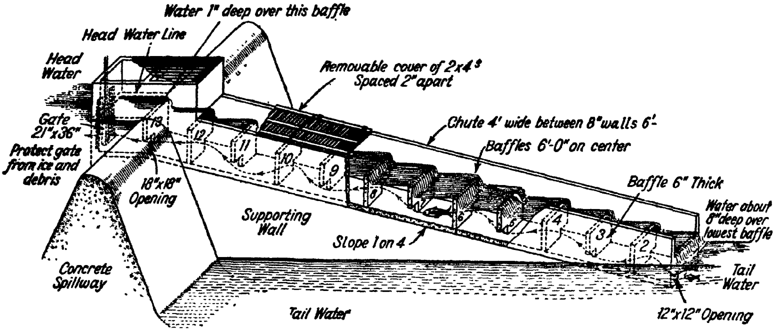


FIG. 188.—View of Reinforced Concrete Fish Chute. Reinforcing not Shown. State of Michigan, Department of Conservation. Make Openings through Baffles Increase in Area Uniformly between the Lowest and Highest Opening.

fore, the entrance to the lower end of the fish ladder should be flush with the toe of the dam, as the fish will not turn downstream to enter it. A slope of the trough of 4 or 5 horizontal to 1 vertical is usual. The required size and type of fish ladder depends upon the type of fish which are to use it. Such details are usually prescribed by the State authorities.

Item 16 of Sec. 148 contains several descriptions of existing fish ladders.

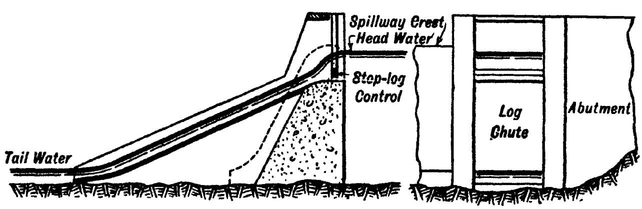


FIG. 189.—A Typical Log Chute.

146. Log Chutes.—Log chutes are required by law in places where there are logging operations on the stream. A typical chute is shown in Fig. 189. It consists of a depression in the crest of the dam, having a bottom elevation 3 or 4 ft. below low-water surface, and provided with a control through which the water passes to a smooth inclined flume. The logs may then pass down this flume to the lower pool. The gradual incline is necessary to prevent the logs from impinging at high velocity directly on the foundation of a shallow

lower pool, with danger of their being split. The inclination of the chute varies greatly with local conditions.

The control, which regulates the flow through the chute, is usually of a type that lowers when more water is required. Drum gates, bear-trap dams, and stop-logs, all previously described, are frequently used for this purpose.

The chute should be made as narrow as possible in order to provide the required depth for passing logs with a minimum discharge. The depth of the water passing the control can be regulated by the operator; but the width is usually not adjustable. The width required depends, of course, upon the size of the logs and the rate at which they must pass the dam. While some log chutes have been provided to pass only one log at a time, chutes 30 ft. wide have also been built for enormous logging operations.

The discharge required to drive logs is sometimes a serious drain on the flow available for power, principally for those streams which are so completely regulated that log-driving operations cannot be confined to flood periods. The discharge necessary to float the logs past rapids in the river may be far in excess of that required to get them by the dam. In such cases the chute must have sufficient capacity for that purpose, unless other means are provided to draw water from the pond.

Timber booms are necessary to guide the logs head-on toward the entrance of the chute.

For large fluctuations of head-water surface, very elaborate controls are necessary. For some such cases, the chute is pivoted at the lower end, and the upper end is raised and lowered according to the elevation of the water surface in the pool.

A description of a very large log chute is given in Item 8 of Sec. 148. Several examples are also contained in Item 16 of Sec. 148.

147. Ice Troubles at Crest Gates.—Ice must be prevented from forming on crest gates, in order to insure their being in operating condition when needed. If neglected in cold climates, ice will form in great quantities below leaky gates, and the entire upstream face of exposed steel gates will be coated with ice several feet thick. The ice which forms on the face of the gates may be too heavy for the hoist to lift and, moreover, the ice will adhere to the piers and sills.

The operation of removing large quantities of ice from the gates is both slow and expensive. In most cases, quick opening of the crest gates is very necessary to prevent flood waters from rising too high and causing damage. Winter floods are frequent in most of the northern states, and even the spring floods may occur before the ice at the gates has melted away. Aside from the great danger of possible unsuccessful operation of frozen gates when badly needed to pass sudden floods, it will be found economical to make provisions to prevent the ice from forming if the gates must be operated during the winter season.

Reference should be made to the Nov. 4, 1924, issue of *Power*, which contains a synopsis of an excellent paper read by Mr. J. S. Bowman before the American Society Civil Engineers, on ice troubles at spillways. This paper has been freely drawn on in this section.

Three methods are used successfully to prevent freezing of crest gates. These are, (1) heating by steam, (2) heating by electricity, and (3) the provision of air jets to circulate the head-water.

For the first method, the downstream face of the gate is housed in, usually with two layers of 1-in. sheeting with building paper between, and steam coils are located close to the skin plate of the gate and also embedded in the concrete near the gate seals. Steam must be supplied continuously during cold weather because, if the pipes are allowed to cool, condensation may freeze them solid when steam is first turned on. The 25-ft. by 15-ft. Taintor gates of the Consumers' Power Co., in Northern Michigan, required 15 tons of coal each for continuous winter operation. The boiler capacity was 15 h.p. per gate.

In some cases, space heaters have been substituted for the steam pipes inside the housing, with equal success. Four 500-watt Westinghouse space heaters are placed at the bottom of each of the 16-ft. by 12-ft. Taintor gates of the Jim Falls dam in Wisconsin, at a total cost of $170 per gate. The heaters require about 5000 kw.-hr. per gate per year.

The W. G. Fargo Engineering Co. has made a practice of heating only about every fourth gate of a series. When the heated gates are opened during the winter, the flow of warm water from the bottom of the pond, passing the balance of the gates, will, within from two to four hours, melt the ice on them sufficiently to permit their being opened.

In the third method of preventing ice formation, compressed air, released from pipes near the bottom of the gate, creates a circulation which draws the warmer water from the bottom of the pond to the surface. This method will not only prevent the formation of ice on the face of the gate and the piers, but will keep an area of open water above the gate. It was first used in this country at the Keokuk development of the Mississippi River Power Company.

The following description of the compressed-air installation at Twin Branch was furnished by Mr. J. S. Bowman, Hydraulic Engineer, Fargo Engineering Co., Jackson, Mich.

In 1923 an air system was installed at the Twin Branch Dam of the Indiana & Michigan Electric Company from designs of the Fargo Engineering Company. The spillway gates consist of seven Taintor gates, 25 ft. long and 10 ft. 6 in. high. This system was installed not so much to prevent pressure against the gates as to prevent the formation of ice on the upstream face and at the ends and sill.[4]

An old 6-in. by 6-in. air compressor, used for dusting the generators, was speeded down to 120 r.p.m. and the air passed through a combined cooler and receiver and a reducing valve to a 1½ in. main on the runway over the gates. At each end of a gate is a ⅜-in. needle valve and union for attaching the ⅜-in. aeration pipes which discharge the air into the corner formed by the sill and the pier. The discharge of the air at this particular point was found to be very important. The aeration pipes are in place only during the winter. Air is supplied by the compressor at a pressure of 15 or 20 lb., and reduced to a pressure of 5 lb. in the main.

[4] This statement has reference to the compressed-air installation at Keokuk, which was designed to prevent ice thrust against the gates.

Approximately 1.0 cu. ft. of free air per minute was used at each of the 14 outlets during the past winter. The temperatures during the month of January were very low for this locality over a protracted period. On nine days the minimum temperature ranged from 0° F. to −16° F., and on four days the daily mean was below zero. For the entire month the average minimum temperature was 10° F. and the average maximum temperature was 29° F. with a mean of 19° F. Ice formed on the pond to a thickness of 16 in., and at the Elkhart plant, eleven miles upstream, where all conditions are similar, ice formed on the face of the gates to a thickness of 12 in. However, the air system was kept in operation at Twin Branch without interruption and with practically no attention, keeping the gates entirely free from ice. The current of water came upward at the junction of the gate and the pier, then circulated on the surface toward the center of the gate. In each corner an area of about 2 sq. ft. remained entirely open, and along the face of the gate the surface was open for a width of from 2 to 12 in.

The power required to operate this system was not over 2 kw. for the seven gates. At a cost of $0.010 per kw.-hr. this would cost approximately $0.07 per gate per day with very slight attendance by the operating force.

The ice must be kept clear of the face of the gate, not only to permit operation, but also to prevent thrust from the ice sheet against the gates. This thrust has been known to damage the gates to a considerable extent. Care should be taken not to thaw the ice away from the projecting nose of the piers, which should take the ice thrust.

Flash-boards require constant attention throughout the winter. Unless the ice sheet is kept cut back from the upstream face, the thrust will cause failure. If the flash-boards leak or if water is allowed to trickle over them, large accumulations on the downstream face may prevent temporary flash-boards from bending over as desired, or permanent flash-boards from being removed. The writer believes that the compressed-air installation previously described has not been used for flash-boards; but it should prove equally successful for this type of crest control.

148. Bibliography.—The following are references to typical examples of head-water control and accessories for dams.

1. Movable Crests for Dams. Eng. Record, Vol. 69, p. 708. (Used by Reclamation Service.)
2. Three Low-head Hydro-electric Developments in Michigan. Eng. Record, Vol. 56, p. 462. (Drum gate.)
3. Floating Crest Gates used on Sherburne Lakes Dam. Eng. News-Record, Vol. 80, p. 124.
4. Improvement Work on River Murray in South Australia, by R. C. Cutting. Eng. News-Record, Vol. 85, p. 244. (Movable trestles and needles.)
5. Stop-Log Barrier Regulates Romanov Canal, by L. Ross. Eng. Record, Vol. 74, p. 114. (Winch for lifting stop-logs.)
6. The Menominee and Marinette Hydro-electric Development. Eng. Record, Vol. 63, p. 36 (15 ft. by 11 ft. Wooden Taintor gate).
7. A Low-head Power Development on the Tippecanoe River. Eng. Record, Vol. 60, p. 612 (12-ft. high Taintor gate).
8. Island Lake Storage Dam Has Immense Log Sluice. Eng. News-Record, Vol. 79, p. 217 (20 ft. by 30 ft. bear-trap dam).
9. Design of Rolling Dams, by A. G. Hillburg. Eng. Record, Vol. 68, p. 654.
10. Rolling Dams for Long Lake Development. Eng. Record, Vol. 70, p. 322.
11. Swedish Government Builds Hydro-electric Plant above Arctic Circle: Part I. Eng.-Record, Vol. 72, p. 156. (Rolling gate.)

12. Rio Guaso Irrigation Dam with Automatic Shutters. Eng. Record, Vol. 63, p. 523. (Collapsible permanent flash-boards, 3.28 ft. high.)
13. Flash-board Design and Experiences, McCall Ferry Dam. Eng. Record, Vol. 66, p. 628. (Temporary wooden flash-boards.)
14. Panels of Movable Weir Collapse Automatically. Eng. News-Record, Vol. 82, p. 818. (Collapsible permanent flash-boards, 5 ft. high.)
15. Balanced Automatic Sluice Gate for Park Dam. Eng. News-Record, Vol. 82, p. 1166. (Collapsible permanent flash-boards, 4 ft. 6 in. high.)
16. Report of Hydraulic Power Committee, 1924, National Electric Light Association. (Description of various types of crest control and sluice gates, fishways and log chutes.)
17. Automatic Spillway Gates of Block Canyon Dam, by Julian Hinds. Eng. News-Record, Vol. 94, p. 1046. (Drum type.)
18. Some Experiences with Large-capacity Reservoir Outlets, by J. M. Gaylord. Eng. News-Record, Vol. 81, p. 945.
19. High-pressure Gates in Dams for Water-works and Irrigation Reviewed, by D. W. Cole. Eng. News-Record, Vol. 81, p. 880.
20. High-pressure Reservoir Outlets, by Gaylord and Savage. U. S. Reclamation Service Report (139 pages), 1923. Published by Chief Eng. Bureau of Reclamation, Denver, Colo.

CHAPTER XVI

CONDUIT INTAKES

By William P. Creager and Joel D. Justin

149. General.—Intakes to conduits are located at the upper end of the conduit or portion of the conduit and are used to contain apparatus for screening débris and ice from the water and gates for controlling the flow.

There may be several intakes to a system of conduits. Thus in the Ocoee No. 2 development, Fig. 103, a fully equipped intake was located at the diversion dam to screen the water and to regulate the flow into the flume. An intake was also provided at the upper end of the penstocks, with gate control for each penstock and screens to remove material that might be taken in along the flume line. Two intakes were also used in the Sewells Island Development, Fig. 108, for the same reason; but the racks were omitted from the intake at the dam.

Intakes for hydro-electric projects may be divided into two general classes: *high-pressure intakes* and *low-pressure intakes*. High-pressure intakes are used, in general, where the drawdown is very considerable, as, for instance, in a reservoir that may serve both as a storage reservoir and as the headwater of a hydro-electric development. This type of intake is more fully described in Sec. 173. Low-pressure intakes, which are the more common class, are used for relatively smaller drawdowns such as may be expected in the daily and weekly water-surface variation in power ponds. They are described more fully in Sec. 172.

There is no clear line of division between the two classes of intakes; moreover, the general characteristics of each class vary so widely with local conditions and the personal preference of the designers that many different types of structures have been used. The author will therefore endeavor, in the following discussion, to give a description of the general features and requirements which are common to all types, and to point out, in a general way, the differences and adaptability of the various types.

The general requirements of intakes may be listed as follows:

(*a*) *Structural Stability.*—The structure must be stable. Low-pressure intakes are frequently an extension of or a part of, a dam, and are subject to all the requirements for dams. High-pressure intakes, if in the form of an unsupported tower, must be stable against ice thrust and earthquakes.

(*b*) *Limitations of Velocity.*—The velocities through the racks, gates, and other passages must be confined within economic and practical limits.

(*c*) *Hydraulic Efficiency.*—The shape of the water passages must be such

that the transformation of static head to conduit velocity is gradual and entails the smallest practical eddy losses.

(*d*) *Practicability of Operation.*—The intake must be adaptable to practical operation. All apparatus should be reliable and reasonably quick of operation. Delays in the repairs to conduits and turbines, due to breakage of intake gates and hoists and other faults, are very frequent. The reader is referred to the 1924 Report of the Hydraulic Power Committee of the National Electric Light Association on "Control and Outlet Works," for a very good synopsis of the requirements of intakes and typical examples.

150. Forebay.—The forebay is the enlarged body of water just above the intake. It may be the pond formed by the diversion dam, or it may be the enlarged section of a canal which is spread out to accommodate the required width of the intake.

It is usually necessary to provide a deflecting device, preferably at an angle

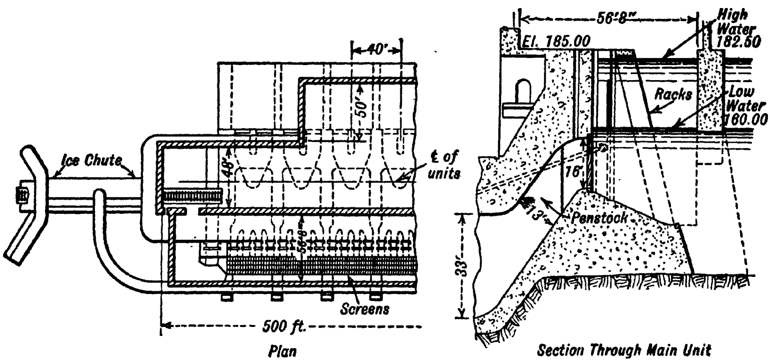


FIG. 190.—Showing Use of Outside Baffle Wall at Holtwood (McCall's Ferry), Susquehanna River, Pa., for Diverting Ice.
(From Eng. Record, Vol. 56, p. 319.)

of 30° to 45° to the direction of flow, to divert ice and trash away from the intake to the spillway or to a sluiceway at one end of the intake.

Figure 190 shows a deflecting device consisting of a fixed concrete baffle wall supported on piers. This type is very expensive, particularly if the forebay is deep and the climate cold, as the baffle wall must be designed for stability against ice thrust.

More frequently the deflecting device consists of a floating boom. Such booms are sometimes made of round logs fastened end to end. It is easy, however, for ice and débris to dive under the boom, and accordingly booms for this purpose are frequently built up of timbers in sections from 16 to 24 ft. long. These sections are fastened together to form a continuous boom. The tension in the boom depends upon the distance the boom projects below water surface, the velocity of the water, and the sag in the boom. For practical purposes, the tension may be obtained by assuming the boom to be an arc of a circle.

Let R = the radius of curvature of the boom, in feet;
α = the angle of the chord of the arc to the direction of flow;
d = the depth of the boom below water surface, in feet;
v = the velocity of the water, in feet per second;
g = the acceleration of gravity = 32.2;
w = the weight of 1 cu. ft. of water in pounds = 62.5;
T = the total tension in the boom, in pounds.

Then,

$$T = \frac{wRdv^2}{g}(1 - \cos \alpha) = 1.94Rdv^2(1 - \cos \alpha). \quad . \quad . \quad . \quad (115)$$

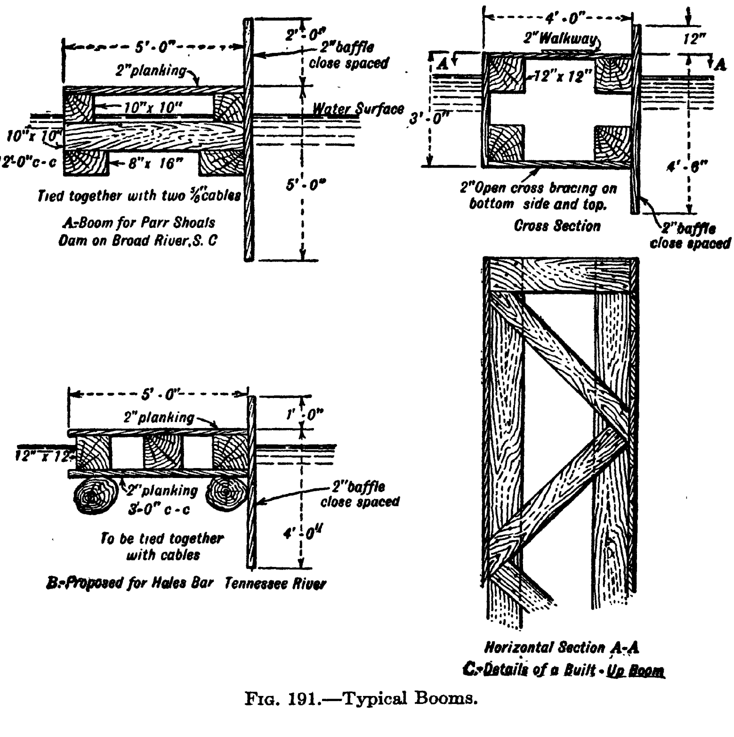


FIG. 191.—Typical Booms.

Ample allowance should be made for the indeterminate effect of wind and the friction of flowing water on an accumulation of ice and débris against the boom, and also for impact of same.

While the impact of ice floes and débris on floating booms usually is not severe, allowance must, of course, be made for unusual conditions. The boom should be loosened from its anchorage after it becomes embedded in ice, if a movement of the ice sheet is to be expected.

Single-log booms are simply fastened at the ends by means of chains; but built-up booms are usually supported on cables or chains which are con-

tinuous between anchorages. Cables are better than chains if more than one is required, because, if there is unequal stress in the cables, those having the greater stress will stretch until the tension is fairly well distributed among them, whereas chains, having less elasticity, would be likely to break under like conditions. Cables should be galvanized or well protected with paint or heavy waterproofing lubricant.

With long booms, the tension may be so great that intermediate anchorages are necessary. These may be provided by concrete or rock-filled timber cribs or, as is frequently done, cables or chains may be attached to the boom at several points and anchored to the bottom of the forebay above the boom. With the latter arrangement, an adjustment of the boom can be made by varying the length of the cables.

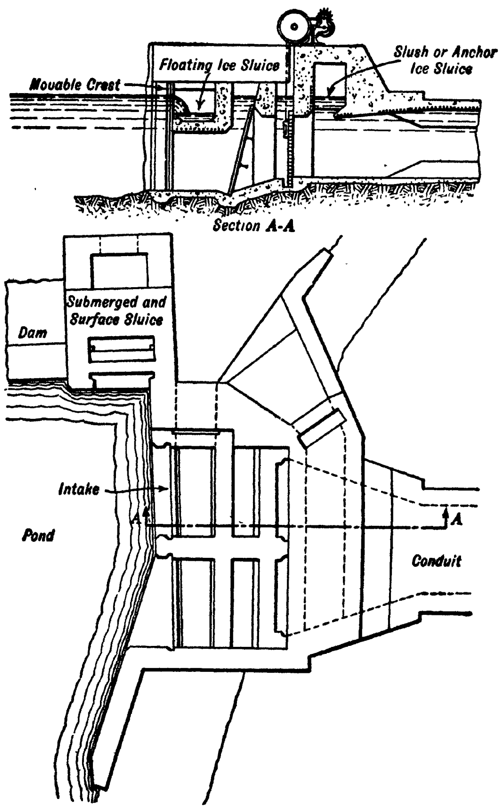


FIG. 192.—Arrangement of Ice Sluice at Plant No. 3, Deerfield River, New England Power Co.
From p. 153, Vol. 67, Eng. Record.

Typical floating booms are shown in Fig. 191. Floating booms have been built in the form of a truss of sufficient strength to span the entire width of the forebay.

Floating booms should be composed of well-seasoned soft-wood timbers and should be painted to prevent water-logging. The ends should be anchored in such a manner that the connection is free to rise and fall with fluctuations in the water surface. The depth to which the boom and its baffle projects beneath the water surface is usually about 3 or 4 ft.

A sluiceway at the outlet end of the boom, as shown in Fig. 192, is essential in order to pass cake ice and trash when no water is passing over the spillway, and to create a greater surface velocity toward the outlet. The sluiceway is usually provided with stop-logs to regulate the depth of outflow.

151. Velocity through the Racks.—Wide variation exists in the velocities at the racks of various developments; but, in most cases, the velocities have been confined between 1.5 and 2.5 ft. per second through the gross area of the racks. The gross area, as here used, is the vertical area of the water passage at the racks, including the area of racks and supports but not the area occupied by concrete piers.

As the loss of head at the racks for usual velocities is very small if the racks are kept clean, the permissible velocity is not determined by the economic loss of head but by the maximum velocity at which the racks can be conveniently raked.

The following velocities through the gross area of the racks are recommended for full flow demand in ordinary cases:

Velocity at normal low water, not greater than 1.75 *ft. per second;*
Velocity at extreme low water, not greater than 2.00 *ft. per second.*

"Extreme low water," as used above, is that corresponding to an extraordinary and infrequent drawdown of short duration. For intakes at storage reservoirs where the drawdown is great, the velocity at full pond is extremely small. If the intake is on a stream that carries very little drift, leaves, or ice, or if the rack bars are very wide apart, it is logical to permit higher velocities through the racks because, if raking is infrequent, more time can be spent in doing it. Experience has shown, however, that at velocities of about 2.5 ft. per second hand raking becomes very difficult, particularly if the racks are deep. It is claimed that mechanical raking permits the use of higher velocities. It is questionable, however, just how far we should go in providing racks that require a machine to keep them in operating condition. The author prefers to consider machine raking as an additional safeguard, or merely as a more efficient way of raking, rather than to design racks that are known in advance to require machine raking.

For cases where the final protection to the turbine is provided by racks at the lower end of an open conduit, the rack bars at the upper end are frequently very wide apart in order to screen out only that which might plug the intake gates or the conduit. In such cases, much higher velocities are allowable.

152. Velocity through the Gates.—For successful operation, the velocity through the gates, for a given maximum discharge, may vary between wide limits. However, as the higher velocities result in lower gate cost, but greater eddy loss and less power output, there is usually one size of gate which will make for greatest economy of design. The reader is referred to Sec. 87, Chapter X, for a discussion of the general theory of economic design.

Velocities through the gates at full load vary in different developments from 2 to 19 ft. per second, depending on the head on the development, the shape of the load curve, the design of the gate, and other factors. High-head developments permit of high gate velocities as the percentage of head loss is relatively small. It is evident that markets having a peak demand of very short duration will permit of higher full-load velocities than those for which the peak demand is of long duration. Common velocities are 2 to 4 ft. per

second in very low-head plants, 4 to 7 ft. per second in medium-head plants, and 7 to 10 ft. per second in very high-head plants.

153. Hydraulic Efficiency.—It is of great importance in the design and construction of intakes to have the water passages as smooth as possible, to eliminate all unnecessary losses of energy. Changes of section should be made gradually and not abruptly. It will pay the engineer to devote considerable time to a study for determining the smooth gradations in curvature which will give minimum losses. This will frequently mean warped surfaces and a consequently increased cost for form-work, but in most cases the energy gained will pay a large return on this additional investment.

The most desirable and economical curvature to use for these gradations will be one that makes the velocity changes plot on a straight line. That is, if the velocities at various intermediate sections between the prescribed sections are plotted as ordinates and distances as abscissae, the minimum loss will take place if the points plot on a straight line.

154. Intake Gates.—Before reviewing the various types of intakes, it is necessary to describe the requirements of intake gates and valves and the adaptability of the various available types.

Intake gates and valves are readily divided into the following classes:

(a) *Sliding gates*, or those that slide directly on their seats without the interposition of rollers. They are made of various materials but are all of the same general design.
(b) *Roller-bearing gates*, or those that are provided with roller bearings to lessen the friction of opening and closing. These are usually of the following kinds:
    (1) Fixed wheels, as in Fig. 206.
    (2) Detached wheels, as in Fig. 203.
(c) *Pivot gates*, which revolve about a spindle through the center or one extremity of the gate. Gates of this type are of three classes:
    (1) Butterfly valves, Fig. 211.
    (2) Taintor gates, Fig. 215.
    (3) Flap gates, Fig. 218.
(d) *Needle valves*, Fig. 264.
(e) *Cylinder gates*, Fig. 219.

The type of gate to be adopted depends mainly upon the size of the opening, the head on the gate, and the operating conditions, which will correspond to either of the following cases:

(1) The gate must open and close against full operating head with free discharge through the intake.
(2) The gate is required to operate only after the conduit is filled through a small auxiliary filler gate and the head on the gate reduced to a small part of the total head.

Gates controlling long pipe lines are usually of the first operating condition so that they can be closed readily in case of a break in the pipe or turbine

casing. They are frequently equipped with motors operated from the power house. Gates for low-head plants and short pipe lines are more frequently of the second, or filler-gate, class. A more detailed description of the design and operation of filler gates is given in the next section.

Whether one or more gates are used to control a single open or closed conduit is purely a matter of relative economy, and, in many instances, a number of different layouts must be investigated before the most economical arrangement can be decided upon.

There is no standard set of rules which can be given to fix the type of gate most adaptable to given conditions. In general, that type is adopted which, for desired operating conditions and grade of apparatus, can be installed for the least cost.

On account of excessive vibration, slide and roller-bearing gates cannot be depended upon to operate successfully under free discharge into a closed conduit at large part gate opening when the head is in excess of about 50 ft. Therefore, such gates are seldom used under greater heads unless a filler gate is provided to fill the conduit and reduce the velocity through the main gate when it is being opened. For high-pressure intakes controlling long pipe lines, the use of a filler gate complicates the details of the structure and provides for very slow filling of the pipe. Consequently, for such cases, other types of gates and valves are usually adopted.

Taintor gates are used only for very small submergence, as it is very difficult to make them tight at the top.

Flap gates have only been used under low heads with filler gates. This type of gate has been infrequently used of late.

Butterfly valves or needle valves are frequently adopted for high-pressure duty, needle valves being preferred for extremely high heads.

155. Sliding Gates.—This term generally includes all vertical-lift gates, which slide on their bearings in the grooves without the interposition of rollers of any sort. They are usually built of wood, of structural steel, or, for small gates, of cast iron or cast steel. These small gates are frequently bronze-mounted although there is little necessity for their being so unless they are to be kept closed for long periods. The larger steel gates are not usually bronze-mounted, although these and even some wooden gates have been provided with bronze strips to reduce the friction.

Sliding gates, in common with all types that have a continuous bearing under compression all around the opening, provide the minimum of leakage. Sliding gates have been used extensively for various heads, up to a hoist capacity of about 35 tons.

Filler Gates.—Sliding gates are usually the most economical type for low pressures and moderate sizes. However, they require a greater force to operate them than any other type. Consequently, for large sizes and heavy pressures, the capacity and cost of the hoist to operate them becomes very great. For this reason, economy frequently demands a more expensive but easier-lifting type of gate, or the use of a small auxiliary filler gate through which to equalize the pressure while the main gate is being operated.

The filler gate is usually a small sliding gate, sometimes mounted on the

main gate itself, but more frequently on the intake controlling a passageway which connects to the conduit.

The assumption is frequently made that the filler gate must be designed to permit the closing of the main gate under a leakage equivalent to the flow through two of the turbine gates. A smaller leakage is usually assumed for the opening condition, as, prior to opening the intake gates, the turbine gates are accessible for temporary or permanent repairs. Therefore, in some cases, the main gate has been heavily weighted with concrete, as in Fig. 236, on the theory that the weight will assist closure and, with less leakage, the head against which the gate will be required to open will be less than that for closure.

Let A = the gross area of the main gate, in square feet;[1]
a = the gross area of the filler gate, in square feet;[1]
a_1 = the net area of the filler gate, in square feet;
W = the submerged weight of the main gate, in pounds;
w = the submerged weight of the filler gate, in pounds;
K = the coefficient of static friction of the main gate;
k = the coefficient of static friction of the filler gate;
F = the required capacity of the main-gate hoist, in pounds;
f = the required capacity of the filler-gate hoist, in pounds;
q = the leakage under which the main gate must close, in cubic feet per second;
H = the head, in feet, on the filler gate necessary to pass the discharge, q;
h = the mean head on the filler gate at high water with empty conduit, in feet;
C = the coefficient of discharge of the filler gate;
g = the acceleration of gravity = 32.2.

The required capacity of the filler-gate hoist is,

$$f = 62.5\,hak + w. \qquad (116)$$

As already explained, the required capacity of the main-gate hoist is determined for the condition of closure. Assume the turbine gates to be closed as far as possible, but the conduit still discharging the assumed leakage, q. The main gate is lowered, after which the filler gate is closed. As the main gate closes, the difference in pressure on the two sides of it gradually increases, owing to the gradual reduction of the opening, and the leakage flow through the filler gate increases. The maximum difference in pressure or head on the main gate occurs when the gate is nearly closed and the whole amount of leakage is passing through the filler gate. Obviously, this head is exactly that necessary to pass the leakage through the filler gate, and may be derived as follows:

From Eq. (21)

$$q = C$$

[1] Usually taken as equal to the net opening plus one-half the bearing area.

Or,

$$H = \frac{q^2}{2gc^2a_1^2}. \quad \ldots \ldots \ldots \quad (117)$$

The required capacity of the main-gate hoist is:

$$F = 62.5HAK - W. \quad \ldots \ldots \ldots \quad (118)$$

Or, by substitution from Eq. (3):

$$F = \frac{62.5AKq^2}{2gc^2a_1^2} - W. \quad \ldots \ldots \ldots \quad (119)$$

Equations (116) and (119), giving, respectively, the required capacity of the filler and main-gate hoists, show that the larger the size of the filler gate, and hence the greater the capacity of the filler-gate hoist, the smaller is the required capacity of the main-gate hoist. The relative capacities of main and filler-gate hoists, for a given example, are shown in Fig. 193. It is usually assumed that the greatest economy obtains when the two hoists are of the same capacity, which, for the example, is 7500 lb. corresponding to a filler gate 12 sq. ft. in net area.

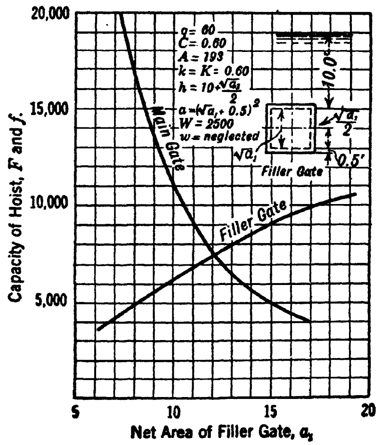


FIG. 193.

When the size of the filler gate and the capacities of the hoists have been fixed, it is necessary to determine, for the capacity of main-gate hoist adopted, under what leakage the main gate can be opened. This determination is as follows:

Assume both the main and filler gates closed and the conduit empty. After the filler gate has been opened and the conduit allowed to fill, the hydraulic gradient in the conduit will rise to a level such that the head on the filler gate is reduced to a value, H_1, which is exactly that necessary to supply the unknown leakage, q_1. Obviously, the main gate must now be opened under this head, and the amount of leakage under which the gate can be lifted with hoist capacity, F, previously adopted, is:

$$q_1 = \sqrt{\frac{(F - W)2gc^2a_1^2}{62.5AK}}. \quad \ldots \ldots \quad (120)$$

In the previous example, the leakage under which the gate can be opened, as determined from Eq. (120), is 48 cu. ft. per second, or 80 per cent of the 60 cu. ft. per second under which it can be closed.

There are two objections to the use of filler gates:

(*a*) The main gate cannot be closed if the leakage is greater than antici-

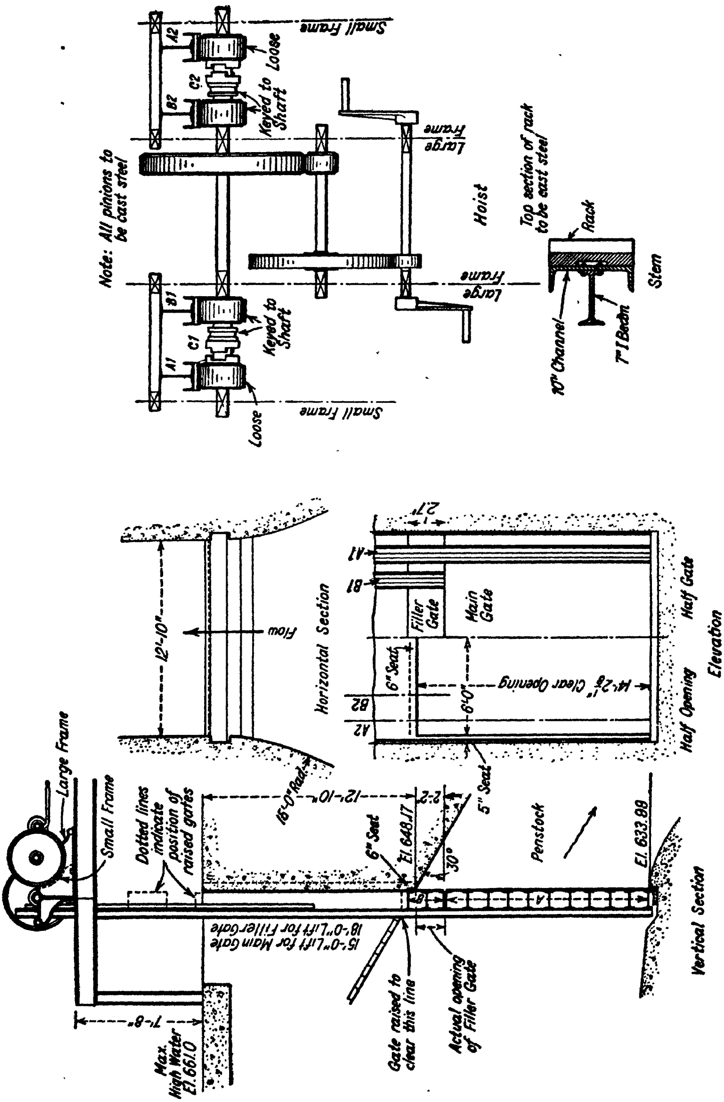


FIG. 194.—Creager Duplex Hoist for Deferiet Development, Power Corporation of New York.

pated, and it is then necessary to place stop-logs or other emergency means of stopping the flow, at a considerable expense and loss of time.

(*b*) The filler gate, being of small size, will not fill the conduit quickly and is not adaptable to remote control.

For this reason, filler gates are seldom used in modern developments having long conduits, although they are very frequently employed in very low-head plants with short conduits, particularly if they control the opening to a concrete spiral-case turbine, where the danger of excessive leakage, except through the turbine, is practically *nil*.

Figure 194 shows the layout of the Creager Duplex Gate, in which the main and filler gates are operated by the same hoist.

This gate is made in two parts, namely, the main gate, A, and the filler gate, B. The two gates are not attached to each other, and each has a separate pair of stems. The stems for the main gate are indicated as $A1$ and $A2$ and the stems for the filler gate as $B1$ and $B2$. The main gate and the filler gate operate independently of each other, but by the same hoist.

For the hoist, the pinions $B1$ and $B2$, for the filler gate, are permanently keyed to the shaft so that the filler gate must rise or lower at all times when the gate hoist is being operated.

The pinions $A1$ and $A2$, for the main gate, are loose on the shaft, and the gate hoist will not raise or lower the main gate unless the clutches $C1$ and $C2$ are thrown in to engage the pinions $A1$ and $A2$.

The entrance, which is assumed to be closed, is opened by the following method:

The clutches, C, are disengaged from the pinions, A, and the gate hoist is operated to open the filler gate, B. After the penstock has been filled with water, the clutches, C, are thrown in to engage the pinions, A, and the hoist is again operated to raise both the filler gate and the main gate to the position shown by the dotted lines on the section. The clutches are then in the correct position for lowering.

When the gate is lowered the hoist is operated until the main gate reaches its lowest position. Then the clutches, C, are made to disengage the pinions, A, and the filler gate is lowered to its closed position. For the condition of leakage through two broken turbine gates, the forces necessary to operate the main and the filler gate are each equal to 10 tons.

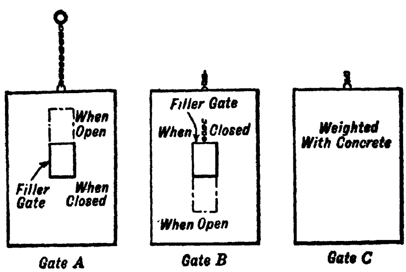


Fig. 195.—Gates for One Turbine Unit, Herrings Development of the Power Corporation of New York.

Figure 195 is a diagram showing the arrangement of the filler gates for one turbine unit of the Herrings Development. A single traveling hoist operates all the gates in the power house, in which there are three turbine units having three gates each. The hoist also operates the filler gates. All gates are of steel construction.

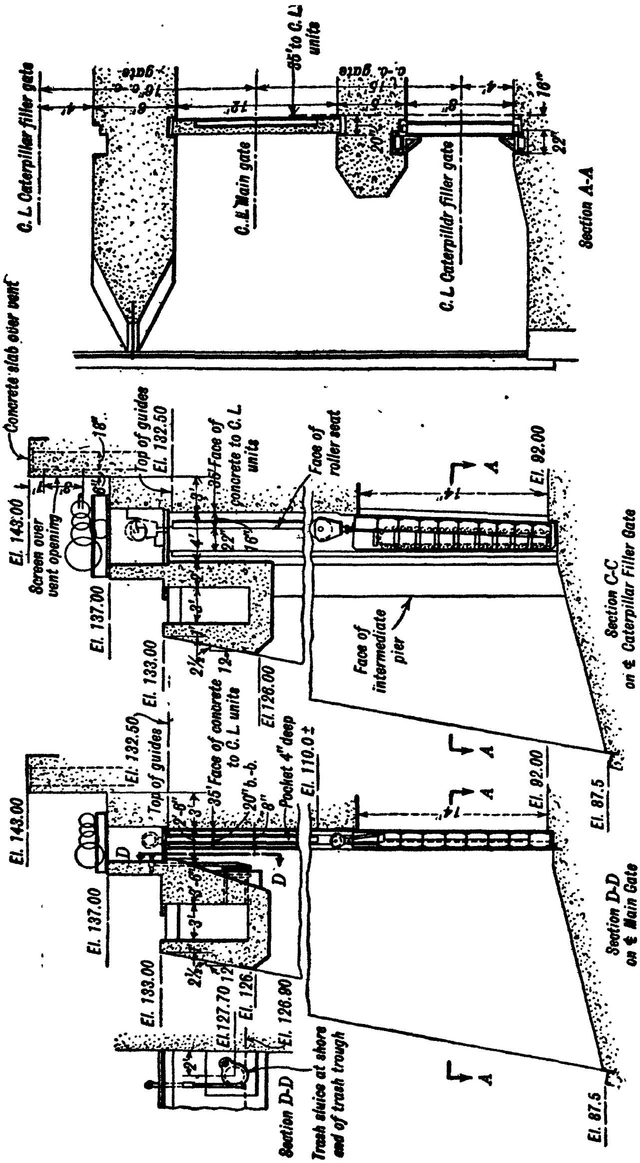


FIG. 196.—Intake Gates, Farming River Power Co.

Gate A contains a filler gate which opens by lifting, and gate B a filler gate which closes by lifting. Gate C contains no filler gate.

When all the main gates are held in the normal open position, filler gate A is closed and filler gate B is open. To unwater the wheel chamber, main gate B, having the open filler gate, is lowered first. Gate A is then lowered, and gate C, being weighted with concrete, is lowered last. Filler gate B is then pulled shut and is held in that position by the pressure of the water after the wheel chamber is drained. Should the leakage be extraordinary, due to a bad break, the heavy gate, C, can be tripped and allowed to fall by gravity.

To open the gates, filler gate A is opened by lifting and held open by fastening its chain to a fixed support. When the turbine chamber has been filled, gate A is lifted and, as it rises, its filler gate closes by gravity. When this gate is up, it is held in position by a latch. The filler gate B, being held in a closed position by the pressure of the water, opens by gravity after the water pressure has been equalized. Gate B is then opened, and gate C, being heaviest, opens last. It will be noted that, after the gates are up, the filler gates automatically take the correct position for closing as previously described.

The upper gate of the Cedars Development, Fig. 243, is the filler gate, and is provided with a powerful hoist for both thrust and pull. The lower gate is operated by cable only, as it is lowered and lifted only when the upper gate is open and the pressure equalized. A similar arrangement is shown in Fig. 196.[2] The filler gate is of the low-friction, caterpillar, roller-bearing type, and the main gate is weighted with concrete. The assumed leakage for gate closure, for this case, is full turbine discharge.

Timber Sliding Gates.—Timber gates are not considered as permanent as steel gates, particularly if they are only partly submerged in the open position. However, as they are very easily replaced and the first cost is much less under low heads, they are very frequently used.

The most popular wood for timber gates, in the East, is Long Leaf Yellow Pine. The Western states furnish Douglas Fir and California Redwood. Short Leaf Yellow Pine, White Pine, and White Oak have also been used extensively.

The thickness of gate timbers may be proportioned according to the usual rules of timber design. The bearing pressure must be limited to that which will not cause brooming under friction. Table XXXVI from the "American

TABLE XXXVI

BEARING PRESSURES IN POUNDS PER SQUARE INCH TO PRODUCE SIDE-GRAIN INDENTATIONS OF 0.01 INCH IN TIMBER

White oak	3310
White Pine	1180
Long leaf yellow pine	2350
Douglas fir	1730
Short leaf yellow pine	1730
Red pine (Norway)	1170
Spruce and Eastern fir	1200
Hemlock	1250
Cypress	1330
Cedar	1030
Chestnut	1770
California redwood	970

[2] E. A. Dow's Paper, A. S. M. E., Spring Meeting, 1925.

Civil Engineers' Handbook,[3]" gives the pressure, in pounds per square inch, to produce a side-grain indentation of 0.01 in. A bearing stress not to exceed 25 per cent of these amounts is recommended for gate bearings.

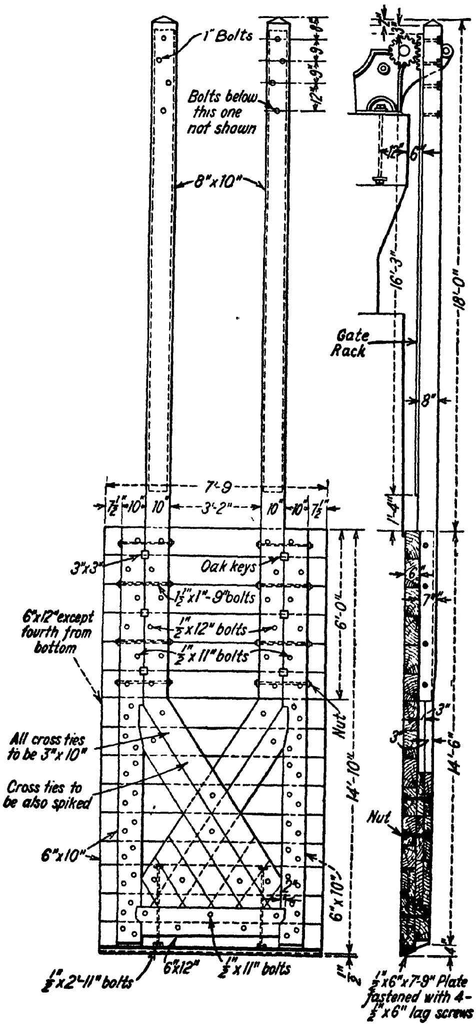


FIG. 197.—Timber Gate for West Buxton Plant, Maine.

Figure 197 shows details of a typical wooden gate with wooden stems, and Figs. 198 and 199 show wooden gates with steel stems. The timber gates are

[3] Mansfield Merriman, Editor-in-Chief, John Wiley & Sons.

frequently provided with splines [4] for tightness and have stiffening planks bolted or spiked to them. The stems are always bolted to the gates. To prevent the upper and lower timbers (particularly the lower) from becoming loosened, it is customary to bolt the first few timbers together by a vertical through-bolt, as in Fig. 199, and sometimes such bolts are continuous from top

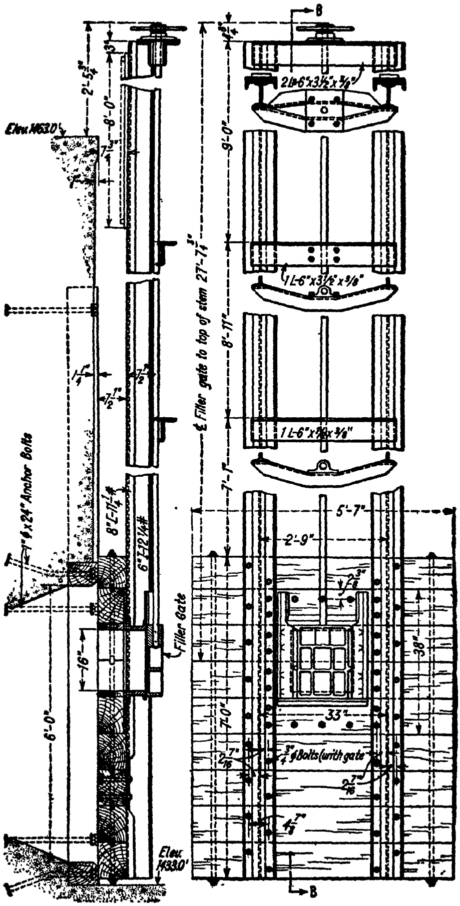


Fig. 198.—Timber Gate for Ordonez Plant, Quito Electric Light and Power Co.

to bottom of the gate, as in Fig. 198. Fig. 200 shows several types of bottom seals for slide gates. Sketch *A* is a simple bearing of the gate on a timber sill, where tightness is not necessary. In sketch *B*, tightness is obtained by a metal strap which acts as a cutting edge and buries itself in the wooden sill by

[4] See sketch (*e*), Fig. 273.

the pressure of the hoist. This type is not adaptable to cases where the hoist is operated by a motor that has a limit switch to limit its travel, unless the

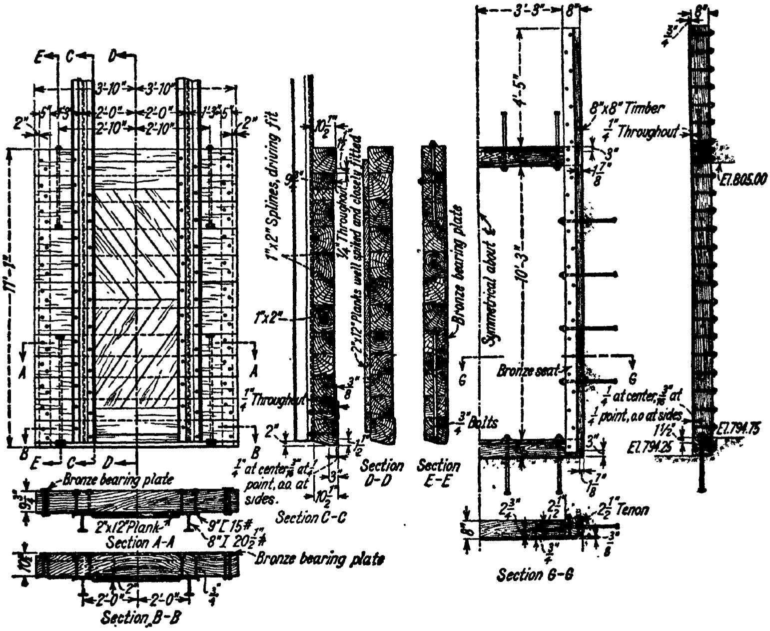


FIG. 199.—Timber Gate for Ocoee, No. 1 Plant, Tennessee Power Co.

last few inches of closure are made by hand. Sketch *C* is a lap bearing on a stepped sill. This type is less efficient hydraulically but is more adaptable to motor operation.

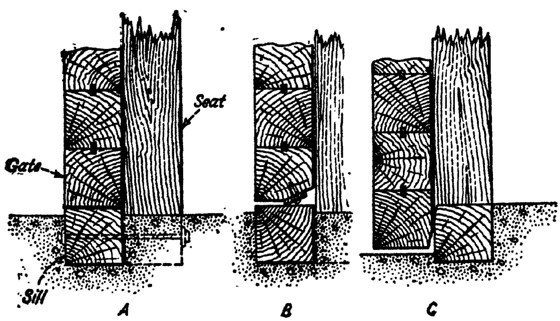


FIG. 200.—Typical Sills for Wooden Sliding Gates.

Gate seats may be wood or structural-steel shapes. Wood seats are permanent if the normal position of the gate is open and the seats are wholly

below water surface. Rapid rotting may be expected at the water line. Steel seats are not machined for smoothness. Wood seats are usually of oak or other hard wood.

Typical steel gate seats are shown in Fig. 201. The section shown is continuous around the opening and fastened at the corners with suitable connections. The side bearings project somewhat beyond the top of the opening.

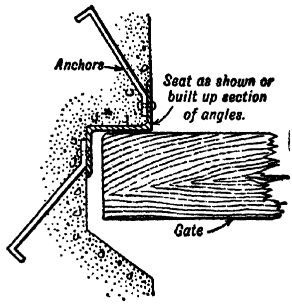


FIG. 201.—A Typical Steel Gate Seat.

Holes for bolts connecting the stems to the gate timbers should be bored for a driving fit.

Steel Sliding Gates.—Steel sliding gates are usually formed of I-beams and channels, spanning the opening with a skin plate on the upper or lower side as best suits the details of the gate.

Figure 202 shows details of a gate of this character, the operation of which is described under "Filler Gates" of this section. This gate is lifted by a chain from an overhead hoist; but frequently steel stems of the type shown in Fig. 199 are used to operate the gate.

Figure 236 shows a combination steel and concrete gate. This gate has

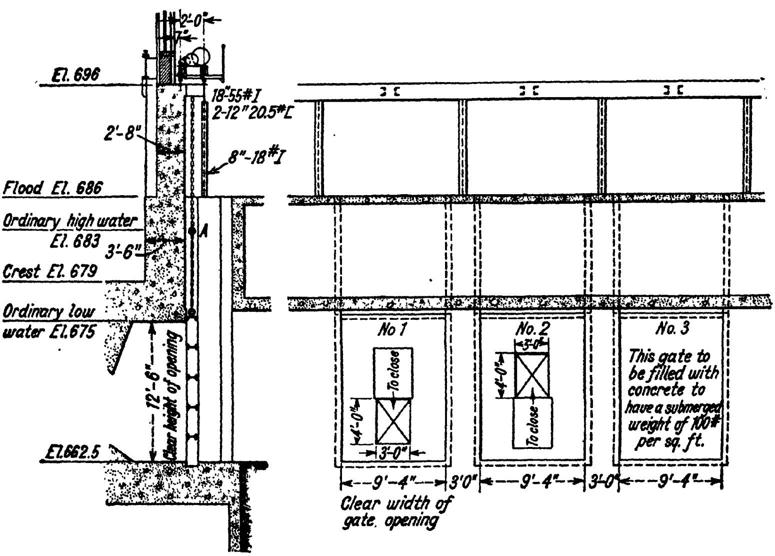


FIG. 202.

no skin plate, the concrete being relied upon for water-tightness. The I-beams slide directly on cast-iron seats.

Cast-iron and Cast-steel Sliding Gates.—For very small openings, such as

are frequently used for small filler gates, cast-iron or cast-steel gates have been used in the past; but it is now the usual practice where metal gates are required, to provide structural-steel gates for ordinary intake purposes.

Stems for Sliding Gates.—Wooden stems are adopted for wooden gates unless the force necessary to operate the gate is such that steel stems are required. Typical wooden stems are shown in Fig. 197. Steel stems for wooden gates are usually made of I-beams and channels riveted together as in Fig 199, and steel stems for steel gates are frequently made in the same way. Wood and steel stems of the foregoing types are used with hoists shown in Fig. 229 and are, in effect, columns fixed at the lower end, eccentrically loaded, and pin-connected at the top.

The equation for the *actual* maximum unit stress in such columns is given by Ketchum [5] as follows:

For flexure and compression:

$$P = \frac{W}{A} + \frac{Md}{I - \left(\frac{WL^2}{KE}\right)} \quad \ldots\ldots\ldots \quad (121)$$

For flexure and tension:

$$P = \frac{W}{A} + \frac{Md}{I + \left(\frac{WL^2}{KE}\right)} \quad \ldots\ldots\ldots \quad (122)$$

Where P = stress, in pounds per square inch;
W = total load, in pounds;
A = area of the stem, in square inches;
M = moment, in inch-pounds, about neutral axis of stem;
d = distance, in inches, from neutral axis to side nearest the load;
I = moment of inertia, in inches cubed;
L = length of stem, in inches, between hoist pinion and top of gate;
K = a constant. Equals 24 for one end hinged and the other fixed, as in the gate stem; and
E = modulus of elasticity of the material of the stem.

The maximum actual stress for each case occurs at the extreme fiber nearest the load. The actual maximum unit stress in tension is always less than in compression, and is therefore ordinarily not a governing condition, as the allowed stress in tension is much greater than that allowed in compression for gate stems.

Equation (121) gives the *actual* maximum unit compresisve stress. This should not exceed the *allowed* maximum unit stress given below.

[5] "Structural Engineers' Handbook," Second Edition, p. 534. McGraw-Hill Book Co., Inc., New York, 1918.

The allowed maximum unit compressive stress in wooden stems should not exceed that given by the following equation:[6]

$$P = F\left(1 - \frac{L}{60D}\right), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (123)$$

where, in addition to the foregoing notation,

F = safe stress in short columns, in pounds per square inch; and
D = least side of wooden stem, in inches.

The allowed maximum unit compressive stress in steel stems should not exceed the following:[7]

$$P = 19{,}000 - 100\frac{L}{r}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (124)$$

where r is the least radius of gyration, in inches. The maximum value of P from Eq. (124) should be limited to 13,000 and the ratio $\frac{L}{r}$ should not exceed 120.

It is good practice, for heavy duty, to specify that the top section of the racks and the hoist pinions engaging them shall be of cast steel.

Should a screw type of hoist be adopted, as in Fig. 232, a round stem must be used; but this type of hoist and stem is very seldom used with wooden gates. The stem should be of steel, as bronze has too low a modulus of elasticity.

Gate stems should be designed for the full starting torque of the gate-hoist motor.

Values of friction coefficients, for use in determining the force required to operate the gate, are given in Sec. 161.

One stem of the round type is usually adopted; but, for the structural-steel type of stems, two are used unless the gate is very small.

For the force required to operate sliding gates, see Sec. 161.

156. Roller-bearing Gates.—Roller-bearing gates are invariably built of structural steel. They are usually designed to operate under full head without the use of a filler gate. Friction coefficients for roller gates are given in Sec. 161. These gates are built in a number of forms, of which the following are the usual types.

Caterpillar Gates.—A caterpillar gate of the Broome type is shown in Fig. 203. This type is used for both high-pressure and low-pressure intakes. It has been used under heads as high as 200 ft.

[6] Am. Ry. Eng. and Maintenance of Way Assn. 1907 Standard. See "American Civil Engineers' Handbook," by Mansfield Merriman, Fourth Edition, p. 762. John Wiley and Sons.

[7] American Bridge Co. Specifications. See "Pocket Companion," Carnegie Steel Co., 22d Edition, 1921, p. 137.

Each side of the gate is equipped with a continuous chain of "caterpillar rollers" which runs on a vertical seat in a slot in the intake piers and travels around the gate as it is raised or lowered. The chain eliminates friction to a very large extent and allows the gate to lower solely by its own weight under free discharge of the opening. Therefore, the drum type of hoist, described in Sec. 167, is used for its operation.

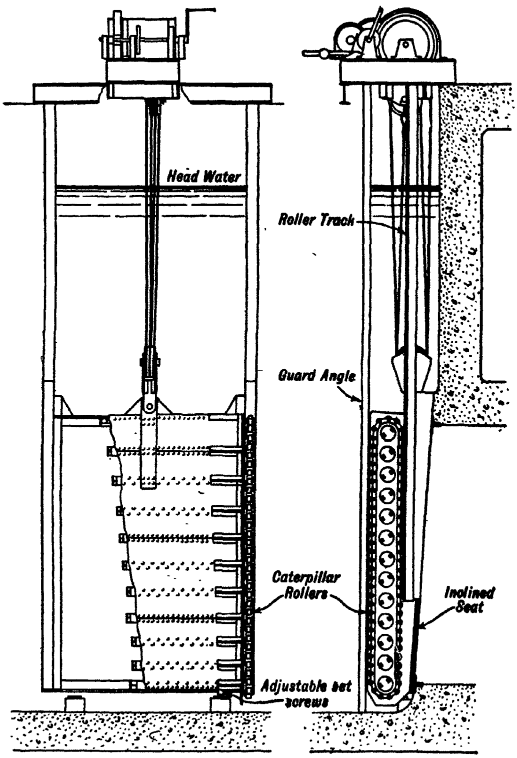


FIG. 203.—Caterpillar Sluice Gate, Philips and Davies, Kenton, Ohio.

The chain consists of a series of steel rollers about 4 in. in diameter and spaced about 5 in. center to center. The links of the chain are of steel and are separated from each other and from the steel rollers by bronze washers. The link pins are of bronze, so that there is no frictional contact of steel on steel, which might corrode and interfere with proper operation. The chain runs on a cast-iron track fastened to the gate.

Besides the method of movement, another feature of the caterpillar gate is the inclined seat. The motion of the gate is vertical. The seating surface of the gate on the sides is inclined at about ½ in. horizontal to 1 ft. vertical. When the inclined seating surface at the sides of the gate comes in contact

with the corresponding inclined seating surface of the frame, the gate is closed and there is no further movement, there being no sliding on the seat of the frame in a properly adjusted gate. A very nice adjustment is required in order to eliminate any sliding or wedging when these two machined seating surfaces come together. This is accomplished by two large adjusting screws set in the lower I-beam. Once the adjustment is properly made, there should be no trouble from wedging of the seats. There is very little leakage through this type of gate. The gates are manufactured under the Broome patents by Philips and Davies of Kenton, Ohio.

Typical installations of caterpillar gates are shown in Figs. 204 and 205. For the force acquired to operate caterpillar gates, see Sec. 161.

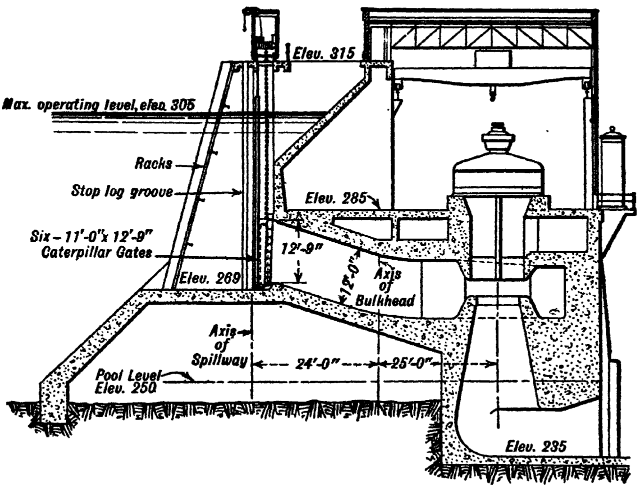


Fig. 204.—Remmel Development, Arkansas Light and Power Co.

Sernit Gates.—The Sernit gate, as made by the M. H. Treadwell Company, Inc., of New York City, is a steel-frame head gate, equipped with large rollers or wheels on the sides of the gate, working in the gate guides. The gate is suitable for use up to about 60 ft. head. When about to seat, the large rollers or wheels roll into a slight depression in the bearing guide so that the bearing strip on the gate is brought into contact with the bearing strip on the guide. The spacing of the wheels on the gate is such that, until the gate is in final seating position, there is never more than one of the wheels over the seating depressions. Consequently, the wheels do not descend into these slight depressions until the gate is in final seating position. When starting to raise the gates, the starting torque is increased by the amount necessary to pull the wheels or rollers out of the slight depressions into which they roll when the gate seats. However, these depressions are only from $\frac{3}{4}$ to 1 in. deep and the angle which they make with the vertical is so small that the additional starting torque required of the hoisting apparatus is slight.

Typical details of a Sernit gate are shown in Fig. 206, and a typical installation in Fig. 207.

The friction of the rollers of the Sernit gate is such that it will not lower entirely under its own weight, and a positive thrust is required to close it. Therefore, a screw-stem type of hoist, described in Sec. 166, is used.

For the force required to operate Sernit gates, see Sec. 161.

Stoney Gates.—Stoney gates have been frequently and successfully used for headgates in low-pressure intakes as well as for flood gates. The gates are structural-steel lift gates which bear on trains of rollers working in the guides.

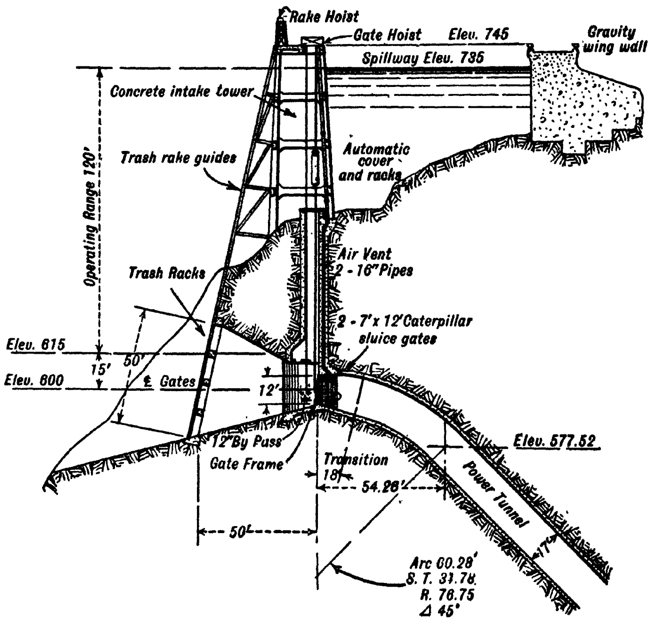


FIG. 205.—Intake and Cushman Dam, Washington.

The roller trains are not connected to the gate at all and consist merely of a large number of hard-steel rollers held together in a light steel framework. As the gate is raised it actually rolls on the rollers (if pressure is back of the gate). When the gate is coming up, the train of rollers moves up also, but only half as fast as the gate. A typical example of the Stoney gate is that used in the Yadkin Falls Development, Fig. 208.

The Stoney gate is capable of lowering solely by its own weight under free discharge of the opening. Therefore, the drum type of hoist, described in Sec. 167, is used for its operation. It is adaptable to practically any size and head.

A satisfactory way to secure the roller chain is to attach a wire rope to the

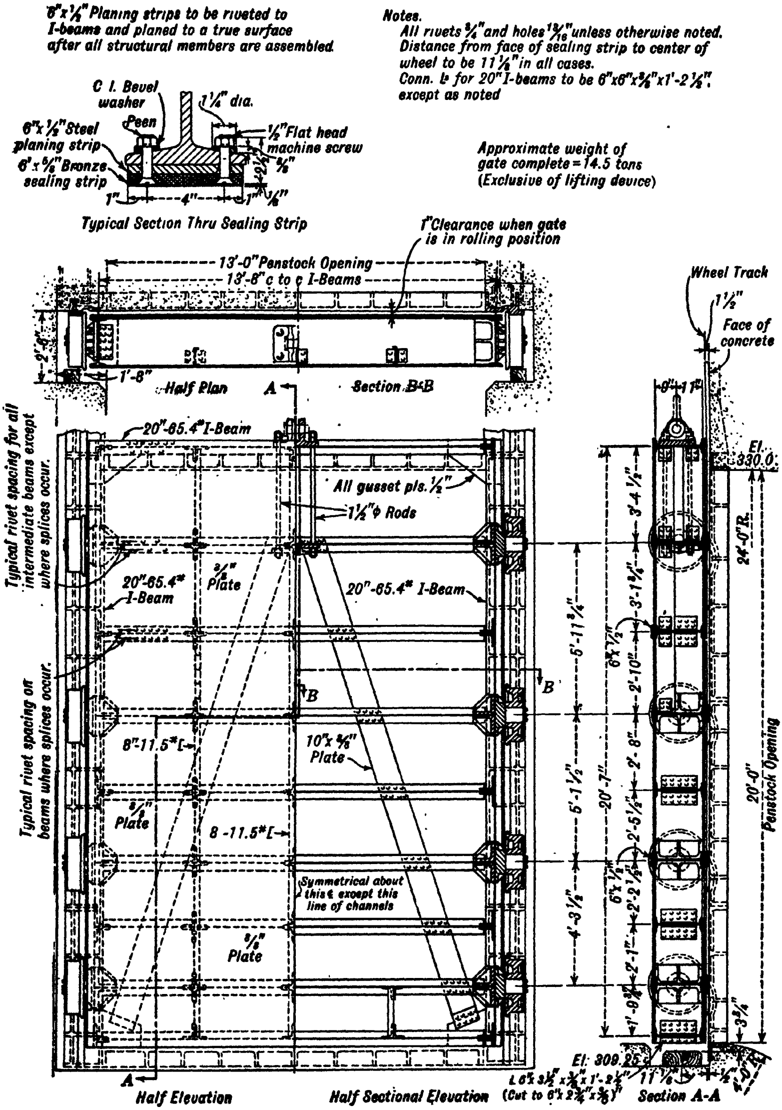


FIG. 206.—Sernit Gate Used for Mitchel Dam Head Gates, Alabama Power Co.

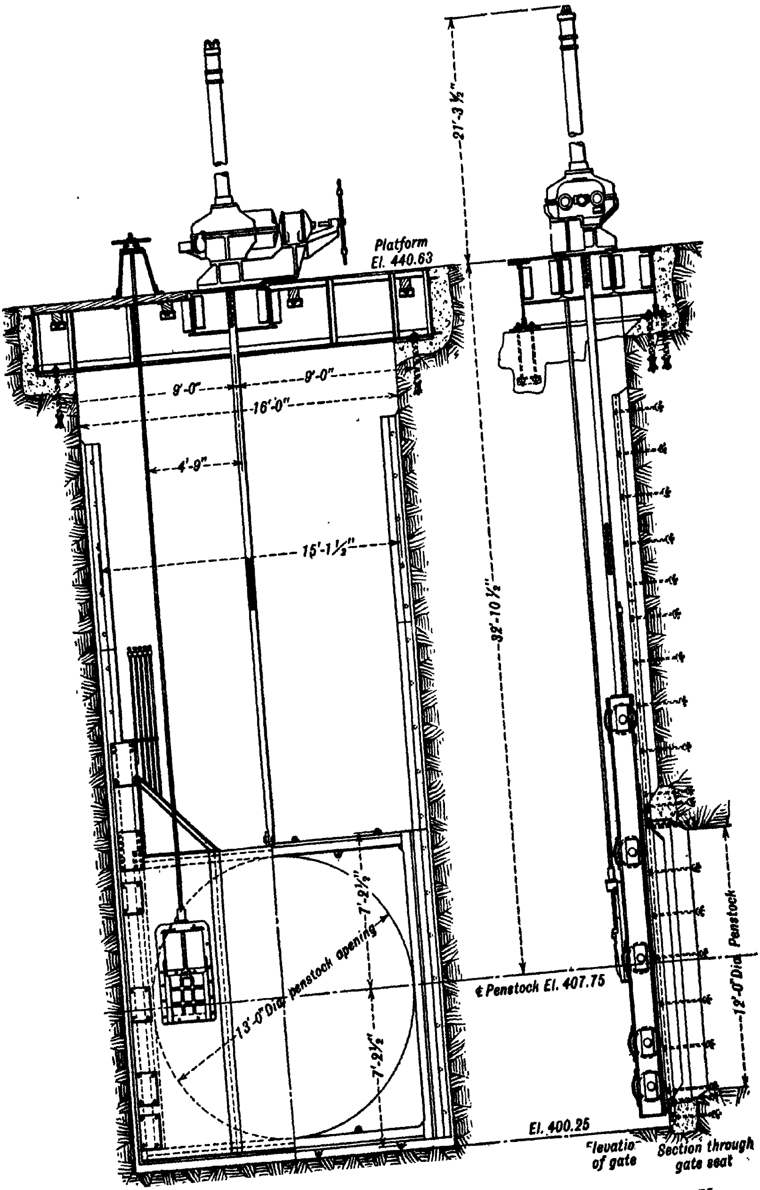


Fig. 207.—Sernet Gate Installed at Spiers Falls, Hudson River, N. Y.

underside of the operating bridge, pass it under a sheave on the roller train, then up to a bracket on the top of the gate itself.

The Yadkin Falls gates were originally intended to be made tight on the sides by the use of a loose pipe, which was to ride with the gate and bear on both the gate and the gate seat. However, the gates were made tight on the sides,

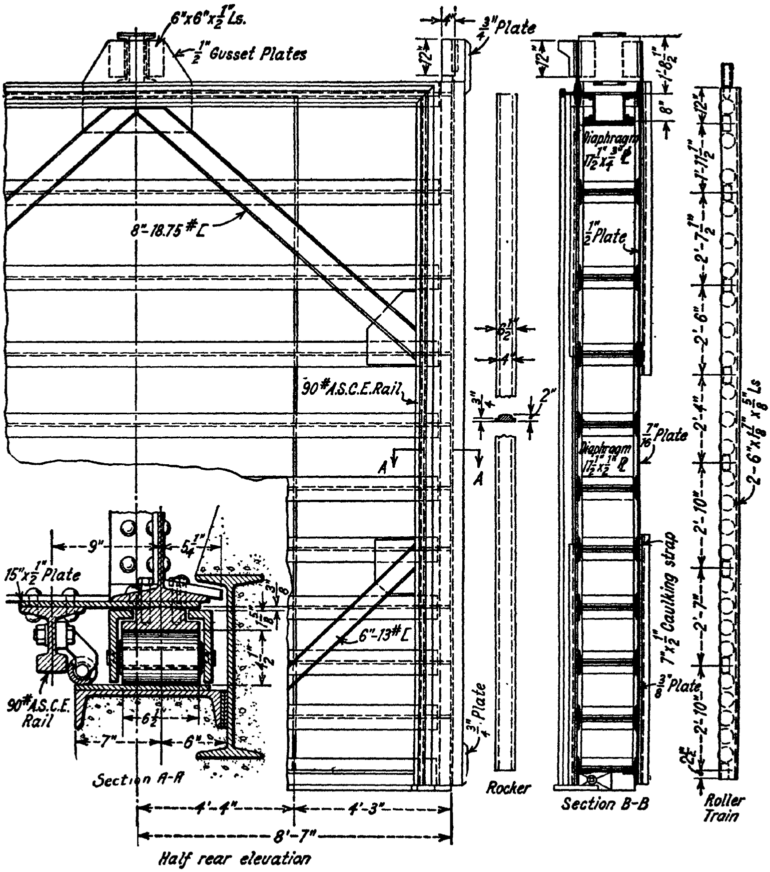


FIG. 208.—Stoney Gate for Yadkin Falls, N. C. Development, Tallassee Power Co.

as shown in Fig. 209, by the use of rubber belting, fastened on the sides of the gate and turned through an angle of 90 degrees so that it rubs along the face of the intake pier just upstream from the guides or, better yet, along the face of a channel or angle set in the concrete. Five-ply best-quality rubber belting should be used for this purpose. The belt should be held to the face of the gate by means of a $\frac{3}{8}$-in. by 2- or 3-in. strip of flat steel, which is bolted to the

face with the belt, between it and the face of the gate, by means of about ⅜-in. bolts spaced about 6 in. center to center.

The bottom of the gate is usually made tight by scribing to the sill a timber set in the bottom of the gate. A more effective way of securing water tightness at the bottom of the gate is by means of a rubber hose inserted in a recess about one-half the depth of the outside diameter of the hose, as shown in Fig. 210. The hose used is ordinary garden hose of about 1 in. diameter. One-quarter inch bolts, having an eye at one end, are put through the buffer timber on the bottom of the gate. The upper end of each of these bolts is threaded and fitted with a nut. At the lower end the eye is allowed to project into the recess in the timber mentioned above. These bolts are spaced about 6 in. center to center. Then slots are cut in the upper part of the hose, and the hose is inserted in the recess in the timber so that the eyes of the bolts project through the slots in the hose. A ¼- or ⅜-in. rod is then run through the hose and through the eyes of the bolts. Then the nuts on the other ends of the bolts are tightened, thus securing the hose in position. The gate sill should consist of a channel or timber set in the concrete and carefully trued and leveled. If care is exercised in the work, this detail will secure an almost perfectly tight seal at the bottom of the gate. It is applicable also to Taintor gates and slide gates.

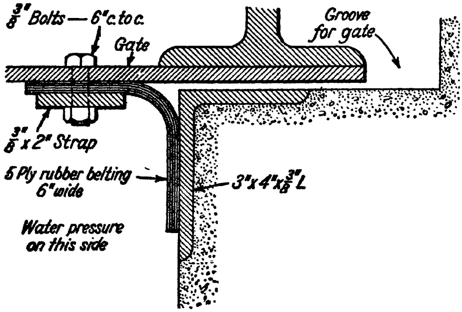


FIG. 209.—Side Seal for Stoney Gate.

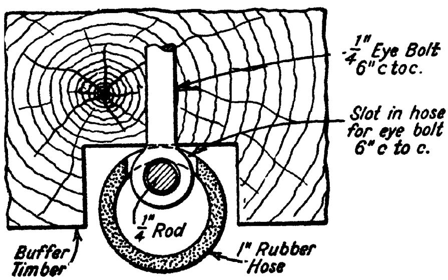


FIG. 210.—Hose Type of Seal for Bottom of Stoney Gates.

Weights of Stoney gates are given in Fig. 210*A*, which is from an article by E. W. Lane in *Engineering News-Record* of December 31, 1925, with additional weights furnished by P. L. Heslop, shown by crosses. A quotation from Mr. Lane's article follows:

In the study now being made of the improvement of the St. Lawrence River for navigation and power development, the question of the cost of Stoney sluice gates came up. A search of engineering literature revealed considerable data on the weights of gates of different dimensions. This was analyzed on the accompanying diagram.

This diagram gives the weight in pounds per 62.5 lb. unit of pressure on the gate, for gates of various spans. To find the weight of a gate, take the product of its width, its height, the average head, or head at the center of the gate, and the value shown from the curve for a gate of the given span. Three curves are shown, with a corresponding formula for weight of gate. The proper curve to use will depend on the degree of conservatism desired in the estimate. In some cases it was not possible to determine from the literature if the weights given included the guides and rollers and values near the conservative curve probably include them. They do not include the weight of towers, hoists or counterweights.

These curves can also be used in estimating the weights of sliding gates built of structural shapes.

For the force required to operate Stoney gates, see Sec. 161.

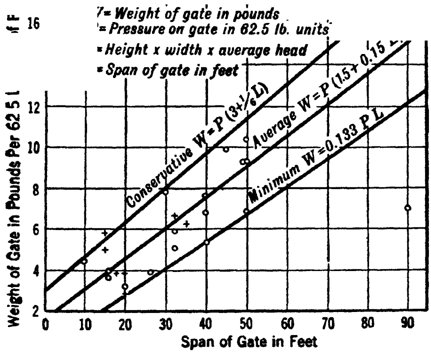


FIG. 210*a*.—Estimating Chart for Stoney Gate Weights.
(Eng. News-Record, Dec. 31, 1925.)

157. Pivot Gates.—Pivot gates for intakes may readily be divided into three main groups.

(*a*) Butterfly valves;
(*b*) Taintor gates;
(*c*) Pivot leaf gates.

Butterfly Valves.—Butterfly and rotary valves, for use in closed conduits, are described in Sec. 190. Such valves are also adapted to intakes. Small butterfly valves are used only under quite high heads; but large structural-steel butterfly valves have been developed up to about 20 ft. in diameter[8] for use under all heads up to about 100 ft. Figs. 211 and 107 show typical examples of the use of a large butterfly valve in a low-pressure intake. Fig. 241 shows the use of butterfly valves in high-pressure intakes.

For the force necessary to operate butterfly valves, see Sec. 161.

The following description of the "Lee" Headgate has been furnished by Mr. Richard Pfaehler.

[8] Allis-Chalmers Mfg. Co., Milwaukee, Wis.

The " Lee " Headgate is of the butterfly, or pivot, type. The center of the gate consists of a cast-steel girder with bronze-bushed trunnions at each end, and the wings of cast iron are firmly attached to the girder by means of machine bolts and shrink rods. By use of this sectional construction, gates of large superficial areas may be built which will develop only slight deflection under water pressure on one side of the gate when used for the closure of large passageways such as penstocks and turbine flumes in hydro-electric power developments.

The gate is so designed that the distance between centroid of the water pressure and the center of the trunnion is slightly larger than is necessary

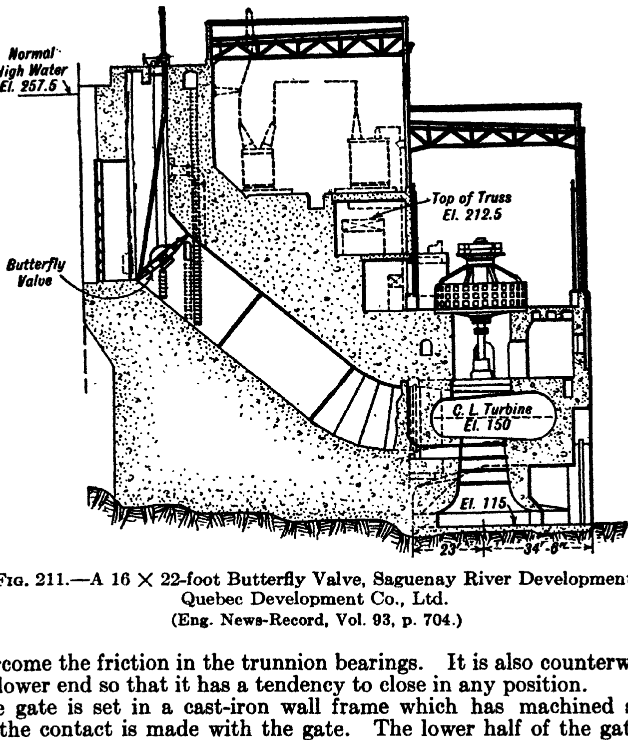


FIG. 211.—A 16 × 22-foot Butterfly Valve, Saguenay River Development, Quebec Development Co., Ltd.
(Eng. News-Record, Vol. 93, p. 704.)

to overcome the friction in the trunnion bearings. It is also counterweighted at the lower end so that it has a tendency to close in any position.

The gate is set in a cast-iron wall frame which has machined surfaces where the contact is made with the gate. The lower half of the gate has a bearing seat on both bottom and sides, while the upper half has a beveled joint and is made water-tight by means of bronze sealing strips.

As shown on Fig. 212, the gate is operated by an individual, completely enclosed, hoist, located above the extreme high-water level.

The screw stem forming part of the hoist is connected to a crosshead moving vertically in a crosshead guide bolted rigidly to a bulkhead wall. The connecting rod to the gate, consisting of an extra heavy steel pipe, is in tension under all operating conditions. To prevent buckling of the connecting rod when the gate is reaching its lowest position, the upper pin connection of the rod is machine-slotted for the pin in the crosshead to allow a 4-in. overtravel on the downward stroke. There is also a nest of heavy coil springs in the gear housing of the hoist, which will act as a cushion for the operating stand in the range of overtravel on the downward stroke and gradually create the neces-

sary overload, causing a tripping of the overload release switch. An allowance of 24 in. for overtravel on the upward stroke is made by proportioning the length of the screw stem and cross-head guide. A gate-position indicator is provided on top of the hoist, also a limit-switch arrangement of moisture and spray-proof construction.

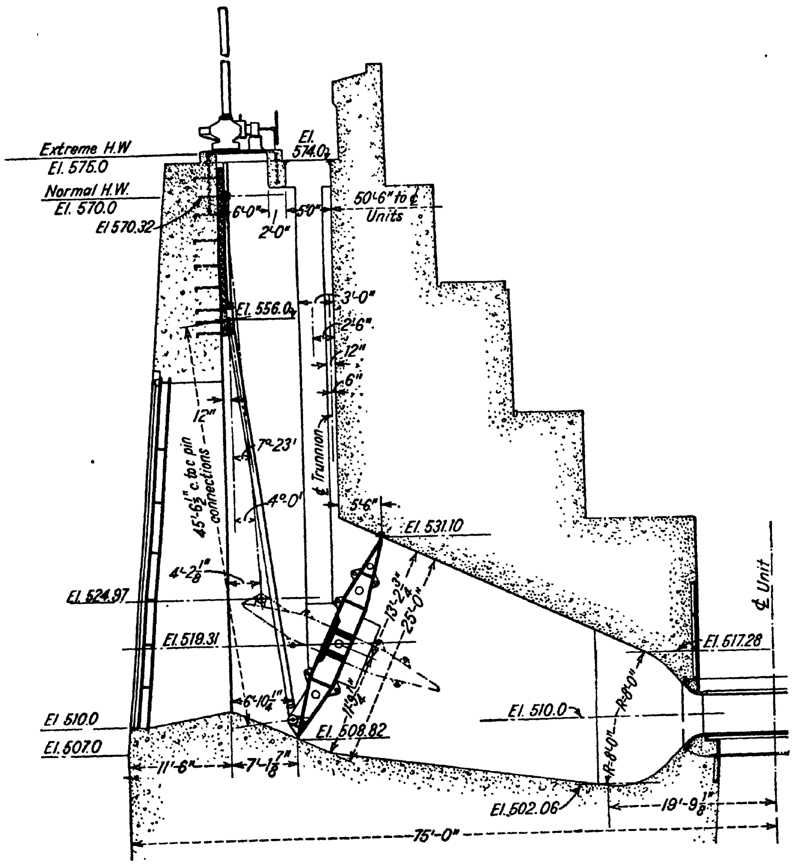


Fig. 212.—Typical Section Showing Arrangement of "Lee" Headgate.

At Mountain Island Station each turbine intake contains two "Lee" gates, and tests made showed the following results:

(*a*) *Closing Gates.*—With the unit carrying full load of 17,500 kw. and the by-pass valves closed, gate No. 1 was closed in eight minutes with a maximum power consumption of 1.66 kw. With the first gate closed and the unit carrying 17,000 kw., gate No. 2 was closed and the maximum power consumption for this condition was 5.5 kw. The head of water on center line of trunnion was 32 ft.

(*b*) *Opening Gates.*—With the intake and scroll case drained and the two gates and by-pass valves closed, the motor was started and the first gate

opened 9 in., requiring a maximum power consumption of 15.5 kw. The scroll case was then filled and this gate lifted to the full open position with a maximum power consumption of 2.48 kw.

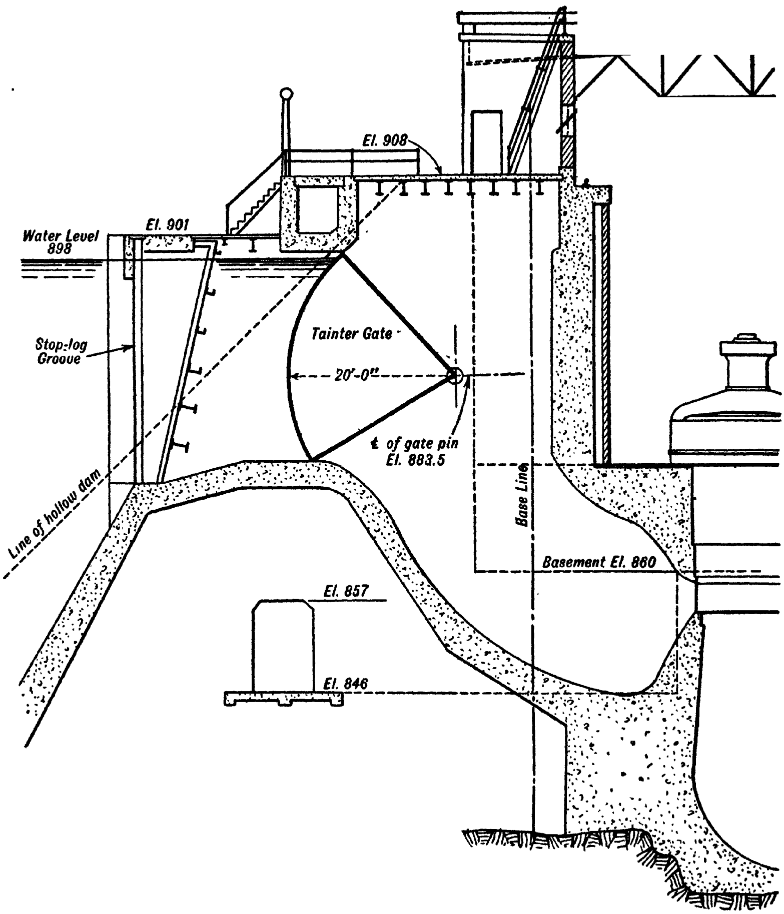


FIG. 213.—Taintor Intake Gate at Wissota Development, Wisconsin.

Table XXXVII gives data on a number of Lee Headgates installed by the Southern Power Co. and allied companies.

Taintor Gates.—These are usually of the type shown in the accompanying figures. They are usually made of steel throughout or steel with wooden facing. For small sizes they have, in a very few instances, been built entirely of wood. This type of gate is most suitable where the fluctuations of water surface are so limited that the top of the gate is at all times at or above water surface. They have, however, been used where water surface is a few feet

TABLE XXXVII

DATA ON LARGE BUTTERFLY VALVES OF SOUTHERN POWER CO.

"The Lee Headgate"

1	2	3	4	5	6	
Station	Size of Gate	Normal Head On Center Line of Gate, Feet	Time Required to Open or Close Gates, Min.	Max. Pull on Connecting Rod,* Lbs.	Motor Torque †	
					Starting, Ft.-Lb.	Running, Ft.-Lb.
Dearborn........	16 ft. wide × 22 ft. high	26	8	78,000	200	80
Mountain Island..	16 ft. wide × 22 ft. high	35	8	120,000	350	140
Rhodhiss........	16 ft. wide × 22 ft. high	31	5	120,000	300	200
New Catawba....	18 ft. wide × 25 ft. high	51	8	150,000	500	380–125
Isle Maligne.....	16 ft. wide × 22 ft. high	53	5	120,000	275	140

* This is for the case of opening the closed gate when subjected to maximum water pressure, considering no back pressure on the gate. Hoists and valves built to operate under this extreme condition.

† For the conditions of column (5).

NOTE: Under normal operating conditions, by-pass valves are used for filling penstocks. Data furnished by W. S. Lee, Chief Engineer, Southern Power Co., Charlotte, N. C.

above the top of the gate. Taintor gates are not very water-tight, particularly at the top of a submerged gate and at the lower corners. The only objection to their use is their lack of tightness when not properly maintained and the space they occupy horizontally.

The gates are true sectors of a circle, and the water pressure, therefore, passes through the pivot. Consequently, they are capable of closing under free discharge by their own weight, and the drum type of hoist, described in Sec. 167, is used to operate them.

TABLE XXXVIII

WEIGHTS OF STEEL TAINTOR GATES

All Dimensions in Feet

Gate	Width (W)	Height (H)	Head to Center Line of Gate (h)	$\frac{W^2Hh}{1000}$	Pounds Weight of Gate
1	25.0	14.0	7.0	61.3	27,100
2	35.0	8.0	4.0	39.2	21,500
3	20.0	13.1	6.5	34.1	19,200
4	15.0	12.2	6.1	16.7	9,700
5	25.0	10.0	5.0	31.2	17,800
6	18.0	12.0	6.0	23.3	12,300
7	6.0	12.0	6.0	2.6	4,300
8	50.0	14.7	7.3	268.0	91,000
9	20.0	20.0	20.0	160.0	46,600
10	25.0	14.0	11.3	99.0	37,600
11	13.0	13.0	27.5	60.5	18,800
12	16.0	9.5	16.7	40.6	21,300

Table XXXVIII gives the dimensions and weights of a number of steel Taintor gates, from the designs of various engineers. The weights do not include hoist chains or cables, or the hoist. They do include the hinges and hinge anchors in the concrete. While some of the gates from which the weights were obtained were provided with wooden faces, the calculated weights were adjusted to include a $\frac{3}{8}$-in. face plate.

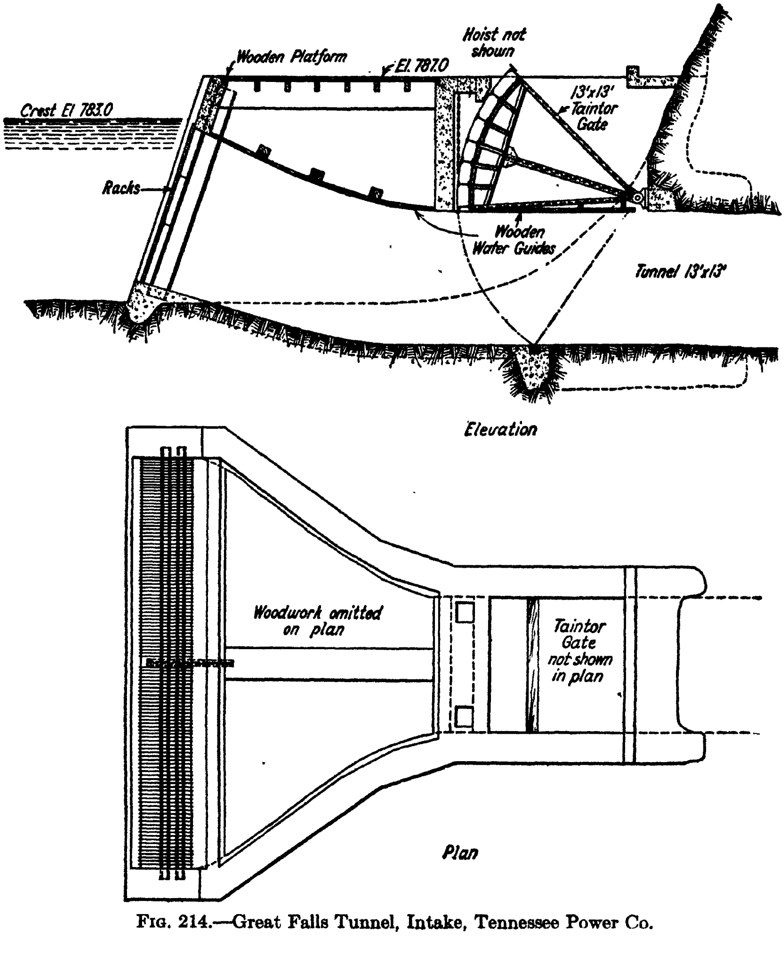


FIG. 214.—Great Falls Tunnel, Intake, Tennessee Power Co.

The width of the gate, W, is the clear width between piers, and the height, H, is the length of the arc of the sector. In other words, $62.5WHh$ would be the total water pressure against the gate.

For the purpose of obtaining approximate weights of Taintor gates without designs, the data from Table XXXVIII have been plotted in the form shown

in Fig. 217. Considering the fact that the gates varied greatly in detail and covered a very wide range of conditions, it is thought that the plotting in Fig. 217 is reasonably uniform. The suggested enveloping curve is therefore recommended for approximate conservative estimates for cases where designs cannot be obtained for more accurate results.

For further details of Taintor gates, see Sec. 139. For the force required to operate them, see Sec. 161.

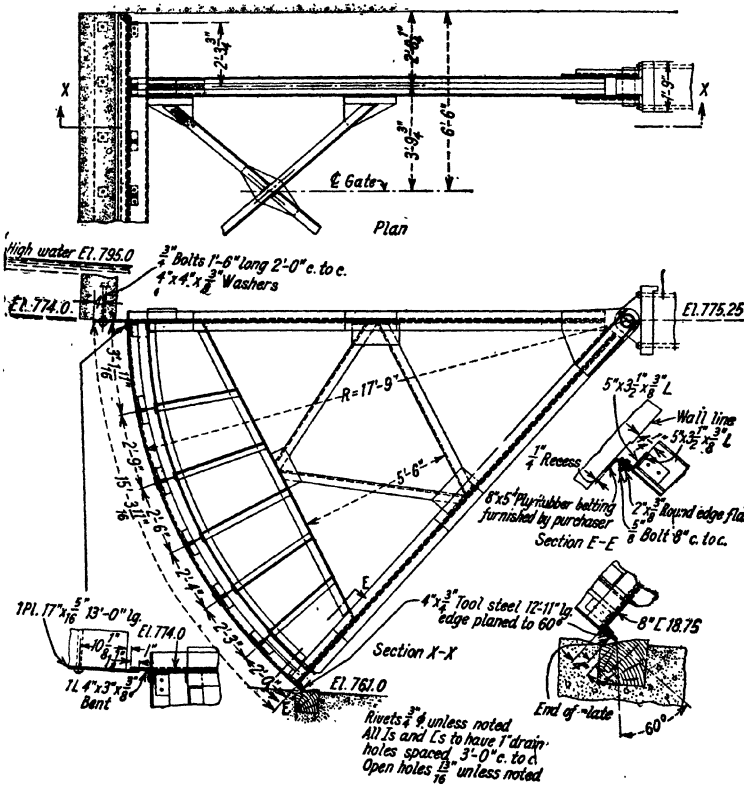


FIG. 215.—Great Falls Tunnel, Intake Gate, Tennessee Power Co.

Pivot Leaf Gates.—A typical pivot leaf gate is shown in Fig. 218. Such gates may be made of wood or steel. As they are normally open, wood construction would be permanent. They rotate about a series of close-fitting cast-iron hinges and are operated, through a chain or cable, by a drum hoist of one of the types described in Sec. 167. They require large filler gates, as the pressure must be very nearly balanced before they can be opened by their own weight; and they cannot be closed under flow without danger of breakage. This type of gate has been used very infrequently of late.

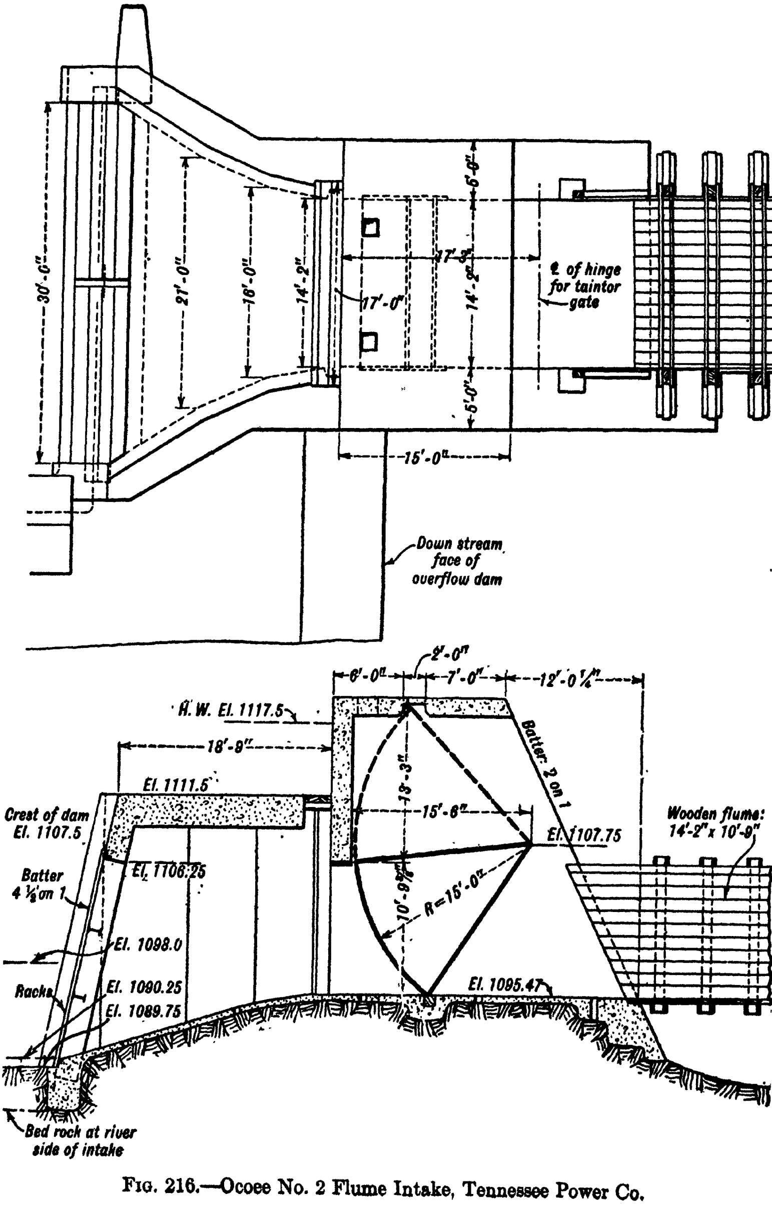


FIG. 216.—Ocoee No. 2 Flume Intake, Tennessee Power Co.

158. Needle Valves.—Needle valves, for use in closed conduits, are described in Sec. 190. Such valves are adapted to intakes under high heads in exactly the same manner as in conduits.

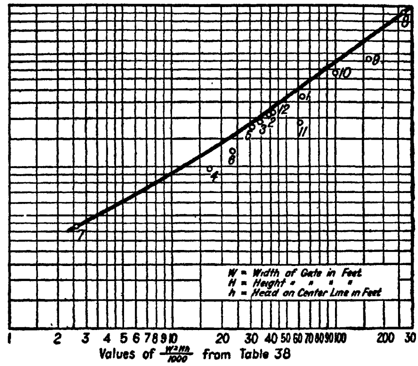


Fig. 217.—Weights of Taintor Gates, Exclusive of Hoisting Chains and Hoists.

159. Cylinder Gates.—A cylinder gate consists of a steel cylinder, open at the top and bottom and having balanced water pressure on the inside or outside surfaces. They are made in a variety of forms, of which those shown in Fig. 219 are examples.

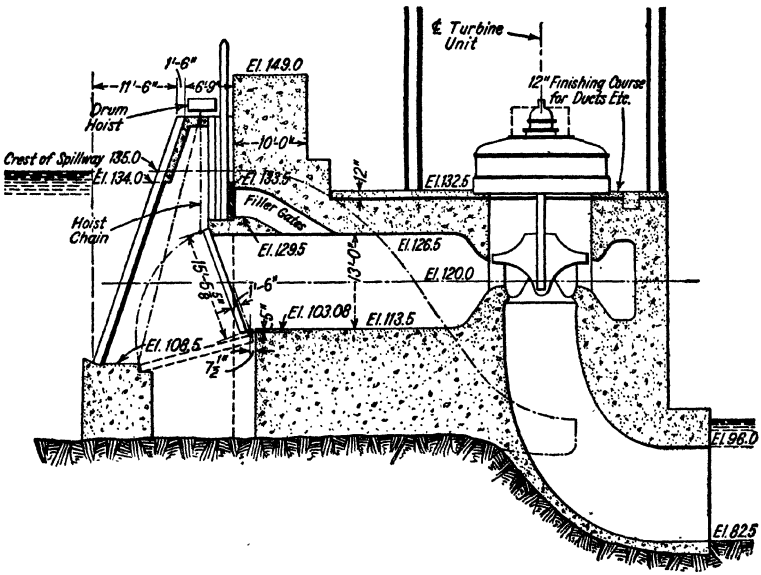


Fig. 218.—Pivot Leaf Intake Gate, Parr Shoals Development, Parr Shoals Power Co.

Sketch *A* shows a cylinder gate with water pressure on the outside. The gate may be lifted by cable or light screw-stem hoist, as the water pressure is completely balanced and the only force to be overcome is the weight of the gate. The bottom of the cylinder rests on the seat *X* and, when the gate is lifted, the water passes through conduit *Y*. The sill at *X* is usually a heavy iron casting provided with a bronze ring on which an adjustable bronze ring on the gate seats. A gate of this type is shown in Fig. 220.

Difficulty is usually experienced in making this type of cylinder gate tight at *Z* and, for this reason, the cylinder is sometimes extended to water surface as shown in Sketch *B*.

In Sketch *C* the water pressure is on the inside of the cylinder, thereby eliminating the necessity of inside stiffeners. The water passes through the passages *M* and *N* by way of the chamber *L*.

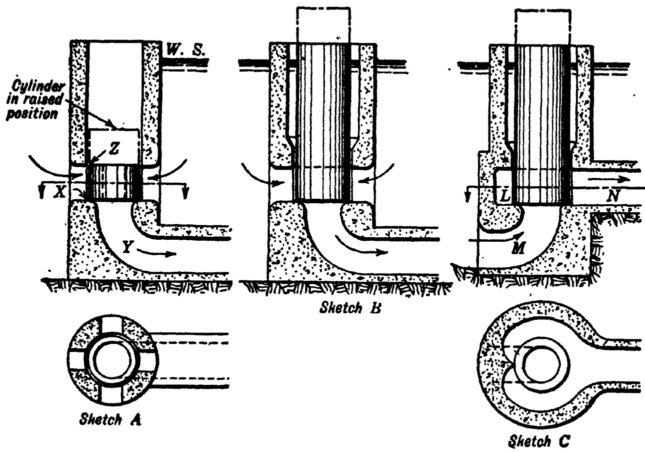


FIG. 219.—Diagrammatic Sketches of Cylinder Gate Intakes.

The following description, with structural details, of two notable examples of cylinder gates, is taken from the "Report of the Hydraulic Power Committee," 1924 National Electric Light Association.

The Southern California Edison Company installed its first cylinder gate in 1920 as an auxiliary spillway gate from the forebay of Kern River No. 3. (This gate is similar to Sketch *B*.)

It consists of a simple steel cylinder 8 ft. in diameter and 22 ft. in height. The lower end of the cylinder is fitted with a bronze seat ring which closes against a similar circular seat ring around the 8-ft. diameter discharge hole in the floor of the forebay. When the gate is seated, the water surrounds the cylinder to the full height. The hydrostatic pressures are thus balanced and the only lifting resistance to overcome is the weight of the cylinder. The lift of the gate is 3 ft. 2 in., and the maximum discharge is 600 sec.-ft.

The cylinder is guided by two pairs of guides, one near the top and the other near the bottom of the cylinder. The guides are diametrically opposite and are recessed to prevent the cylinder from turning. The clearance between

the guides and the guide shoe on the cylinder is ¼ in. The operating mechanism is a rising screw stem attached to the top of the cylinder and operated by an electric motor, remote-controlled from the power-house. The gate has discharged at all openings from a fraction of an inch to full lift, without any

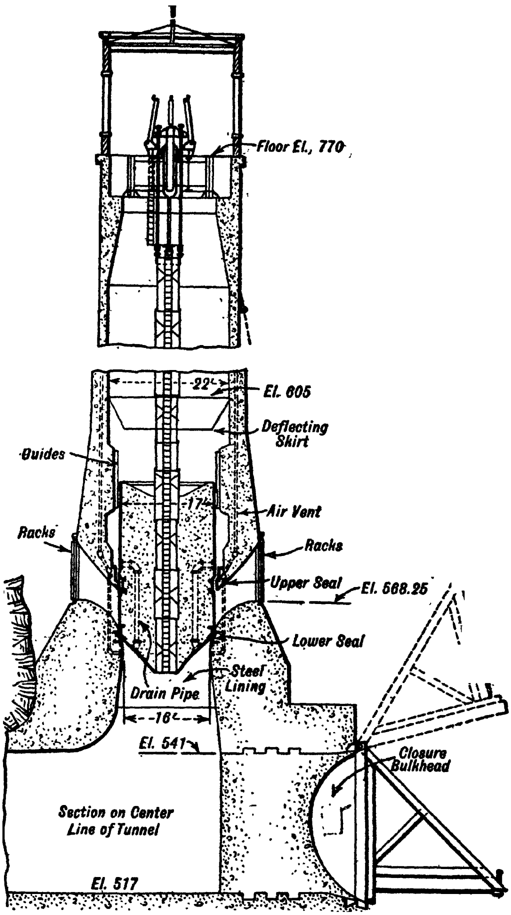


FIG. 220.—Sixteen-foot Cylinder Gate for Dix River Dam.
(Eng. News-Record, Vol. 94, p. 1059.)

excessive vibration. The weight of the gate holds it perfectly tight to the seat.

A second cylinder gate was installed in the summer of this year, at the intake to the 21-ft. diameter tunnel conveying water to Big Creek No. 3. (This gate is similar to Sketch *C*.)

It is 22 ft. in diameter and 77 ft. in height, designed for a maximum discharge of 3000 sec.-ft. It operates inside of a concrete gate tower 28 ft. in diameter.

This type of gate was decided upon after preliminary designs and estimates had been worked up on slide gates, needle valves, and low cylinder gates with top and bottom seats. The reason for the adoption of the type selected was lower first cost, simpler operation and fewer parts subject to damage below water level.

The gate shell is designed to be under tension instead of compression, thus making the stresses in the shell more definite, and avoiding the necessity of heavy interior bracing. This was accomplished by so designing the gate tower that the water passes upward through the gate orifice and then, when the cylinder is raised, passes radially outward into a sort of scroll chamber surrounding the gate, and thence into the tunnel. The gate is not intended to be operated at part openings, the control of the water being by means of gates at the other end of the six-mile tunnel. To avoid possible erosion of the concrete water passages if the gate were partially opened when the tunnel is empty, provision has been made to fill the tunnel through two 3-ft. diameter by-pass gates, the main gate being raised to full opening after the tunnel has been filled with the gates at the lower end closed.

160. Stop-logs.—Provision must be made for unwatering the intake gates for inspection and repairs. In some cases very elaborate provisions have been made to provide emergency gates which can be quickly operated; but such provisions are too special for detailed description. It is considered that properly designed gates should not require frequent unwatering. Consequently, it is usual to provide only for the installation of stop-logs for that purpose.

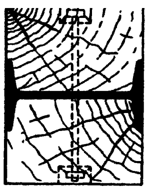


FIG. 221.—Typical Steel-wood Stop-log.

Stop-logs usually consist of timbers which are guided into place by means of a vertical slot at each side of the opening above the gates. Where the opening to be closed is wide and deep, steel I-beams, with timbers bolted to them, have been used, as in Fig. 221. These provide both the necessary strength and sufficient weight to facilitate installation by overcoming the buoyancy of the wood.

Stop-log grooves are sometimes provided upstream from the racks, as in Fig. 213; but much more frequently they are located just upstream from the gate, as in Fig. 204, where the size of the opening is much smaller. In such cases, unless the forebay can be unwatered, the piers between the racks are allowed to project upstream from the racks, as in Fig. 204, so that a bulkhead can be put in place on the extremely rare occasions when it becomes necessary to unwater the racks.

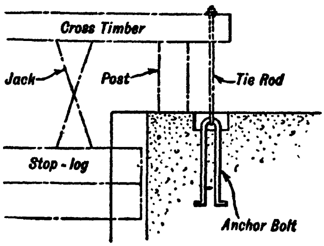
Unless special care is taken to provide exceptionally smooth concrete in the grooves, the stop-logs cannot be put in place and made tight without some difficulty. Consequently, wood or steel seats, similar to those used for sliding gates (Fig. 201), are frequently provided.

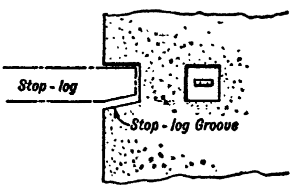
Where wooden stop-logs are used, it is difficult to overcome the buoyancy of the wood without special provisions. Figure 222 shows a permanent anchor bolt embedded in the concrete at each groove. Fastened to this is a temporary cross timber which is used as a support to jack down the logs.

Figure 223 shows an arrangement which is used for those special cases where the buoyancy of the logs is insufficient to bring them to the surface and

where they must be removed against water pressure. It consists of a bolt passing through a notch cut into one end of each log, which may be engaged by a hook on the end of a rod to afford a means of pulling the log to the surface.

161. Force Required to Operate Gates and Valves.—*Sliding Gates.*—The force required to lift a sliding gate is equal to the frictional resistance of the





Plan

Elevation

FIG. 222.—Arrangement for Jacking Down Stop-logs.

FIG. 223.—Special Provisions to Facilitate Removal of Stop-logs.

water pressure on the gate plus the weight of the gate. The force required to lower a sliding gate is equal to the friction less the weight of the gate.

Let F = the force required to operate the gate, in pounds;
A = the gross area of the gate, in square feet;[9]
H = the head, in feet, from water surface to the center of A;
K = the coefficient of static friction; and
W = the weight of the gate, in pounds, corrected for submergence.

Then, to open the gate:

$$F = 62.5HAK + W; \qquad (125)$$

and to close the gate:

$$F = 62.5HAK - W. \qquad (126)$$

The foregoing equations apply to sliding gates used without filler gates. For the force required to operate when filler gates are used, see Sec. 155.

It is recommended that, for wood, rolled steel, and finished castings, when used with similar surfaces or any combination of such surfaces of contact, a value of $K = 0.6$ be used. For bronze surfaces with any of the foregoing

[9] It is common practice, in fixing the value of A, to include one-half the area of the bearing, on the theory that the pressure between the gate and the seat diminishes from full pressure at the outside edge to zero pressure at the inside edge of the bearing.

surfaces, a value of $K = 0.6$ is recommended; but, for bronze on bronze, a value of $K = 0.45$ may be used.

The foregoing recommended values of K are applicable only to the usual case where the intake gates are normally open. If the gates are likely to remain closed for a long period, the foregoing values may be adopted if wood is used for one surface of contact; but, for bronze surfaces on rolled steel and on finished castings, or for bronze on bronze, a value 50 per cent greater than the foregoing values should be used. For normally closed gates, rolled steel and iron and steel castings should never be used except with wood or bronze, as they are likely to rust tight.

A factor of safety to provide for material obstructions under the gate, when closing, is seldom provided, for the reason that such obstructions very seldom occur and, if they do occur, it is much better to remove them than to endeavor to cut through them with extra hoist capacity. Moreover, there is always a margin of strength in every well-designed hoist.

Caterpillar Gates.—According to Mr. E. B. Philips, the hoists designed by Philips and Davies for operating caterpillar gates are figured for:

> The rolled friction of the caterpillar rollers against the roller races, equal to 5 per cent of the hydrostatic pressure against the surface of the gate,
>
> PLUS
>
> The weight of the gate,
>
> PLUS
>
> The vertical component of the water pressure on the inclined surface of the gate,
>
> MINUS
>
> The buoyancy, or uplift, of the gate when closed.

Mr. Philips also states that, although 5 per cent is used above for rolling friction, shop tests show that rolling friction under ideal conditions is less than 1 per cent. This factor of safety, together with 50 per cent or more overload capacity of the hoist, provides for possible inaccuracies in the adjustment of the gates,[10] as described in Sec. 156.

Gates with Wheels.—The force required to operate roller-bearing gates of the Sernit type (Figs. 206 and 207), in which the pressure on the gate is transmitted through axle bearings to wheels which operate on gate guides, is given by Eqs. (127) and (128). These equations apply directly, however, only to the case where the wheels bear on vertical guides and the pull, F, is also vertical. In the case where the guide is at an angle to the direction of pull, as at the beginning of raising the Sernit gate, the force necessary to lift the gate is

[10] Note that 5 per cent rolling friction with a factor of safety of 1.5 is more conservative than the 3 per cent friction with a factor of safety of 2 specified hereinafter for "Gates with Wheels." This gives additional capacity of hoist to overcome the possible inaccuracies of adjustment of the caterpillar type of gate.

equal to $\frac{F}{\cos \alpha}$, where α is the angle between the pull and the direction of travel.

$$P\left(\frac{f_r}{r} + f_a\right) + W, \quad . \quad . \quad . \quad . \quad . \quad . \qquad (127)$$

where F = the force required to raise the gate, in pounds;
P = the water pressure against the gate, in pounds;
r = the radius of the wheels, in feet;
f_r = the coefficient of rolling friction;
f_a = the coefficient of axle friction;
W = the submerged weight of the gate.

$$F_1 = P\left(\frac{f_r}{r} + f_a\right) - W, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (128)$$

where F_1 = the force required to lower the gate, in pounds.

The starting coefficient of axle friction, f_a, with plain axle bearings, has a safe value of about 0.3. If the axle has ball bearings, f_a will not exceed about 0.01. The starting coefficient of rolling friction should be taken as 0.003. (Thurston gives values between 0.001 and 0.003.[11])

In determining the size of hoist, the values of F, determined from the foregoing equations, should be multiplied by a factor of safety of 2.

In this type of gate, careful adjustment must be made to see that the load is not taken by the guides when the gate is closed and the rollers in their respective grooves. If this happens, the force required to start lifting the gate will be the same as that of a slide gate.

Stoney Gates.—With gates of the Stoney type, the pressure is carried on trains of rollers, and axle friction is therefore eliminated. The force required to operate such gates may be estimated by means of Eqs. (127) and (128), by placing the coefficient of axle friction, f_a, equal to zero.

If water-tightness is obtained by a belting strip, the force must be increased by that necessary to overcome the friction of the belting, as explained under the following remarks on Taintor gates.

Taintor Gates.—The force required to lift a Taintor gate may be obtained from the following equation:

$$F = \frac{W_1 r_1 + W_2 r_2 K_2 + W_3 r_3 K_3}{r}, \quad . \quad . \quad . \quad . \quad . \quad . \qquad (129)$$

where F = the lifting force;
W_1 = the weight of the gate;
r_1 = horizontal distance, center of bearing the center of gravity of W_1;
W_2 = total water pressure on the gate;
r_2 = radius of bearing pin;
K_2 = coefficient of friction of the bearing;
W_3 = water pressure on the side sealing strips;

[11] See Mechanical Engineers' Handbook, by Marks, First Edition, pp. 236, 242, and 244.

r_3 = distance, center of bearing to sealing strips;
K_3 = coefficient of friction of sealing strips; and
r = lever arm of lifting cable, as shown in Fig. 224.

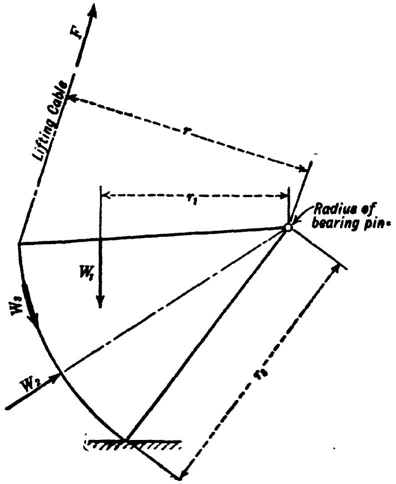


FIG. 224.

All weights and forces in pounds and dimensions in feet.

The friction coefficient, K_2, depends upon the materials of which the bearings are composed, and is given, for each material, in the various books on mechanics. For unlubricated bronze bearings, the value may be taken as 0.45 for gates normally open, and 0.65 for gates normally closed.

The starting coefficient of the sealing strip, K_3, for wet surfaces, from the report of the Hydraulic Committee, National Electric Light Association, 1924, is given as follows:

	New Structural Steel	Rusty Structural Steel	Concrete
Pacific Gas and Elec. Co......	....	0.82 to 0.88	0.56 to 0.70
Joshua Hendy Iron Works.....	0.64 to 0.73		

The pressure, W_3, on the sealing strips should be the total pressure on the area of the strips in sliding contact.

As a matter of fact, the weight, W_1, is usually the preponderating influence on the required capacity of the hoist and, in cold climates, this should include any accumulations of ice which may have to be lifted with the gate.

Butterfly Valves.—There is a closing tendency in butterfly valves at all positions of the disk, except when fully open and closed. Mr. E. A. Dow recommends the following equation, based on tests, for the determination of the moment required to operate butterfly valves discharging freely into the air:

$$M = 7.73hD^3, \quad \ldots \ldots \ldots \quad (130)$$

where M = the moment, in foot-pounds;
h = the head on the valve, in feet; and
D = the diameter of the valve, in feet.

Friction is negligible under heads for which butterfly valves are ordinarily used; but for very low heads it can be allowed for in a manner similar to that described for " Force Required to Operate Taintor Gates."

Normal operation of butterfly valves in closed conduits requires less moment than that of free-discharge valves; but, as it may possibly become necessary to close such valves when there is a break in the conduit, which would cause practically free discharge, the operating mechanism for such valves in closed conduits should be designed to resist a closing moment equal to that given in Eq. (130), in order to prevent it from slamming shut.

The moment required for normal opening of butterfly valves in closed conduits is more difficult to determine and involves a consideration of factors which are too involved for consideration here. As a matter of fact, standard operating mechanism for valves in closed conduits is usually designed for an opening and closing moment as given in Eq. (130) for free discharge.

The value of head, h, for use in Eq. (130) should be the static head on the valve, less all frictional losses above the valve, plus suction head below the valve, if any. This suction head may be built up if the conduit below the valve has a flare, which may reclaim some of the velocity head at the valve as in a turbine draft tube.

162. Capacity and Efficiency of Hoists.—

Let F = the pull or push to be exerted on the gate;
f = the operating force applied to the hoist;
G = the leverage ratio of the hoist, expressed as a decimal;
E = hoist efficiency, expressed as a decimal;
e = efficiency of a pair of gears, bearings included;
T = torque required to operate hoist, in foot-pounds;
S = revolutions per minute of hand crank or motor;
s = speed of gate travel, in feet per minute;
H = Horse power required to operate the hoist;
R and r = radius of hand crank and gears; and
p = pitch of worm or screw gears.

All dimensions in feet and forces in pounds.

The leverage ratio, G, of the hoist, is the theoretical ratio of the force applied to the hoist to the force exerted by the hoist on the gate, neglecting friction. It is also equal to the product of the leverage ratios of its component parts.

Leverage ratios are more easily understood by considering each shaft separately. Thus, in Fig. 225, the leverage ratios are:

$$\text{Shaft } A = \frac{R_1}{r_1},$$

$$\text{Shaft } B = \frac{R_2}{r_2},$$

$$\text{Shaft } C = \frac{R_3}{r_3},$$

and the leverage ratio of the hoist is $G = \dfrac{R_1R_2R_3}{r_1r_2r_3}$.

Also, in Fig. 226, the leverage ratios are:

$$\text{Shaft } A = \frac{2\pi R_1}{p_1},$$

$$\text{Shaft } B = \frac{R_2}{r_2},$$

and the leverage ratio of the hoist is $G = \dfrac{2\pi R_1 R_2}{p_1 r_2}$.

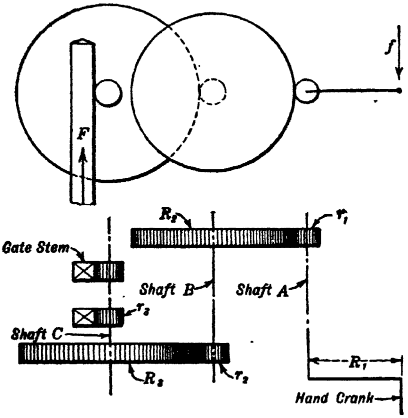


FIG. 225.—Rack and Pinion Hoist with Spur Gears.

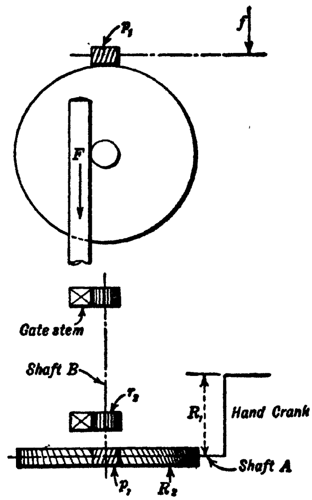


FIG. 226.—Rack and Pinion Hoist with Worm Gear.

Also, in Fig. 227, the leverage ratios are:

$$\text{Shaft } A = \frac{2\pi R_1}{p_1},$$

$$\text{Shaft } B = \frac{2\pi R_2}{p_2},$$

and the leverage ratio of the hoist is $G = \dfrac{4\pi^2 R_1 R_2}{p_1 p_2}$.

Also, in Fig. 228, the leaverage ratios are:

$$\text{Shaft } A = \frac{R}{r},$$

$$\text{Pulleys} = n,$$

where n is the number of turns (4 in Fig. 228).

And the leverage ratio of the hoist is $G = \dfrac{nR}{r}$.

The efficiency of a hoist is the ratio of the theoretical force required to operate the gate, with frictionless bearings and gears, to the actual force

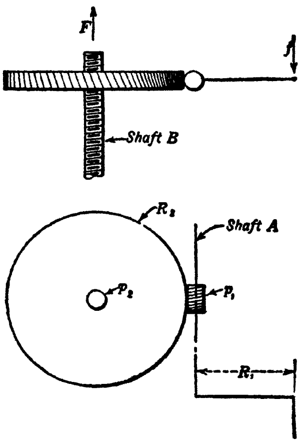


FIG. 227.—Screw Stem Hoist with Worm Gear.

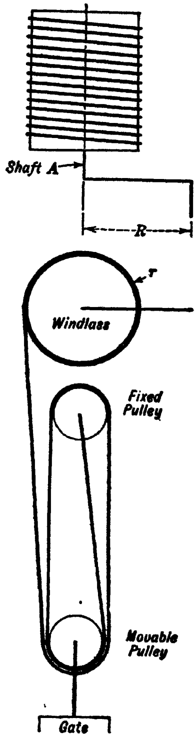


FIG. 228.—Drum Hoist.

required. The efficiency of a hoist is equal to the product of the efficiency of its component parts. Let the efficiency, expressed as a decimal, including bearings and thrust collars, be:

e_R = efficiency of a pair of spur gears,
e_S = efficiency of a screw gear,
e_W = efficiency of a worm gear.

Then, since the hoist of Fig. 225 has three pairs of spur gears, its efficiency is:

$$E = e_R^3.$$

And, since the hoist of Fig. 226 has one worm and one spur gear, its efficiency is:

$$E = e_W e_R.$$

And, since the hoist of Fig. 227 has one worm and one screw gear, its efficiency is:

$$E = e_W e_S.$$

A close determination of the efficiency, e, of the component parts of a hoist can be made only after a careful study of the details of such parts, the proposed method of lubrication, and the care which is to be given the hoist after installation.

Mr. E. B. Philips [12] gives the following approximate values of efficiency of single gears with bearings. These values are considered safe for properly maintained hoists.

	Plain Cast-iron Bearings	Bronze or Babbit Bearings	Ball Bearings	Ordinary Roller Bearings
Spur gears with cut teeth......	0.91	0.94	0.95	0.94
Spur gears with cast teeth.....	0.88	0.90	0.91	0.90
Bevel gears with cut teeth.....	0.90	0.92	0.94	0.93
Bevel gears with cast teeth....	0.86	0.88	0.89	0.88

The efficiency of each steel cable sheave and its bearing is about 97 per cent.

The efficiencies of screw and worm gears vary with the angle of the thread and ordinarily are very much lower than spur and bevel gears. The actual efficiency can be determined only by consideration of many complicated factors, and the reader is referred to the standard textbooks on machine design.

The force, f, required to operate a hoist is:

$$f = \frac{F}{GE}.$$

The torque required to operate a hoist is:

$$T = \frac{FR_1}{GE},$$

where R_1 = the lever arm of the force applied to the hoist.

The number of revolutions per minute, of hand wheel or motor, necessary to operate the gate at a given speed is:

$$S = \frac{Gs}{2\pi R_1}.$$

[12] Of Philips and Davies, machine builders, Kenton, Ohio.

The house power required to operate the gate at a given speed is:

$$H = \frac{Fs}{33{,}000E}.$$

163. Notes on Gearing.—Experience has shown the advantage of spur gearing over all other kinds, in both durability and efficiency. Its chief disadvantage is that it has not the great reduction or leverage ratio that screw and worm gears possess, and that it therefore requires a greater number of gears and more space and weight of hoist.

The inefficiency of screw and worm gears, which require more work to be done in operating the gate, is not noticeable for very small gates or for those hoists which are not hand-operated, particularly if, as is usual, operation is infrequent.

To reduce the size of traveling hoists, worm gears are frequently used in conjunction with spur gears as in Fig. 235.

In some instances, worm gears are used in rack-and-pinion hoists, as in Fig. 230; but the preference for such hoists is for spur gears throughout, as in Fig. 231, as there is usually ample space for a spur-gear hoist.

The efficiency of spur and worm gears is materially increased if they are provided with roller bearings on the thrust collars; but this can be economically done only on the large gears. Both screw and worm gears of the better class of hoists are frequently submerged in an oil chamber, as in Figs. 232, and 235.

Hoists with cast spur gears are usually left exposed to the weather; but the teeth should not be shrouded as water and dirt will be held in them and will freeze. Other classes of gearing should be protected from the weather unless vigilantly cared for.

As obstructions are likely to lodge under the gate, the gearing of the hoist should be designed for the full power of the motive force.

Worm gears are useful to prevent the gate from overhauling the gears and closing rapidly under its own weight.

164. Choice of Type of Hoist.—Mechanisms of various types are used for the operation of intake gates. The usual types are listed below and described in succeeding sections.

Rack-and-pinion Hoists, Fig. 231;

Screw-stem Hoists, Fig. 232;

Drum Hoists, Fig. 234;

Hydraulic Hoists, Fig. 236.

Of these types, the drum hoist is used exclusively for those gates that will close by their own weight, such as the Taintor, caterpillar, and Stoney gates. For those gates that require a positive thrust to close them, the choice lies among the other types.

For very small gates and all gate valves, screw-stem hoists are preferable. For the larger sizes, screw-stem hoists become more expensive and, on account of the low efficiency of the gears, the work required to operate them becomes excessive if hand-controlled. They require very fine adjustment of gate

travel. Any material movement of the stem out of alinement during operation will bind the gear, decrease the efficiency, and cause excessive wear.

Rack-and-pinion hoists are more rugged and require less protection and care. They can be used with stems composed of structural-steel members. On the other hand, they occupy greater space on the intake and are limited as to capacity.

Both types are well adapted to motor control. For hoists between 10 and 30 tons capacity, the rack-and-pinion type of hoist seems to be slightly preferable. The screw-stem type is used exclusively for very large capacities.

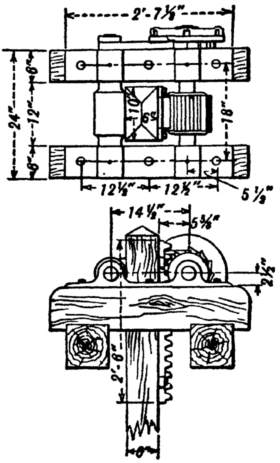


FIG. 229.—Dimensions of Style "A" Hoist, S. Morgan Smith Co., Maximum Capacity 5000 lb.

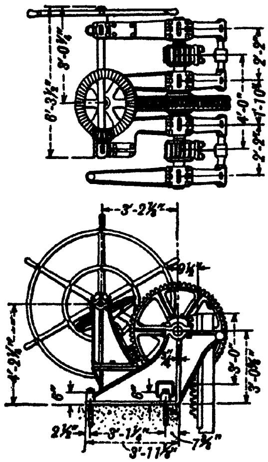


FIG. 230.—Dimensions of the Style "R" Hoist, S. Morgan Smith Co., Maximum Capacity 21,000 lb. for the Two Stems.

Hydraulic hoists have not been used extensively in the past; but, in recent years, many engineers have advocated their use under certain conditions. Their chief advantage is simplicity of construction. On the other hand, they require a number of auxiliaries, such as pumps, tanks, valves, and piping. They are quite well adapted to very large capacity. The near future will probably see more frequent use of this type of hoist.

165. Rack-and-pinion Hoists.—The following are a few of the many types of rack-and-pinion hoists which are on the market.

Single-stem, lever hoist, Fig. 229;

Double-stem, spur-and-worm geared hoist, Fig. 230;

Double-stem, spur geared hoist, Fig. 231.

The lever hoist, in which one end of the lever is frequently a socket wrench fitting on the same shaft as the pinion that operates the rack on the stem, is used only for small capacities.

The teeth of the racks and gears are usually all cast and not cut. This is because of the slowness of operation and the fact that such hoists are fre-

quently exposed to the weather. They are usually of cast iron except that, for the larger capacities, the top sections of the racks, which have the greatest duty, and the pinions engaging them, are preferably made of cast steel. The tendency is to omit all shrouding on the gears, particularly in cold climates, where the hoists are exposed, as it makes a place for the accumulation of ice and dirt, which may cause rupture of the teeth. The omission of the

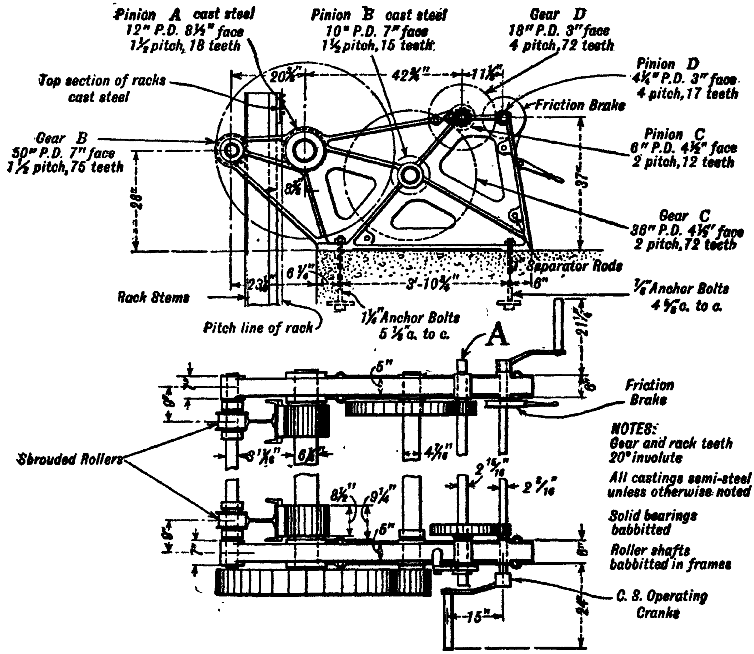


Fig. 231.—Spur-geared Hand-operated Gate Hoist for Two-stem Gate, 60,000 lb. Capacity, Diltz Machine Works, Inc., Fulton, N. Y.

shrouding may make it necessary that the gear, in order to have the required strength, be of cast steel.

If the hoists are to be motor-operated, cut-steel gears should be used for the high-speed portion of the hoist, and such gears should be boxed or housed to protect both gears and motor.

On account of the low efficiency of worm gears and the greater protection and attention they require, spur gears are more frequently used for rack-and-pinion hoists, particularly for the larger sizes, unless lack of space or other special conditions demand worm-gear hoists.

Frequently, provision is made for attaching the operating cranks to a higher-speed shaft, as at *A*, Fig. 231, for speedy operation during light loads. Motor operation is provided by installing a greater gear reduction, and a

clutch should be provided for disengaging this additional gearing for hand operation, in case of failure of the motor.

166. Screw Hoists.—A typical screw hoist of large size is shown in Fig. 232, and a small one in Fig. 233. The former is completely protected from the weather, even to a hood over the rising stem.

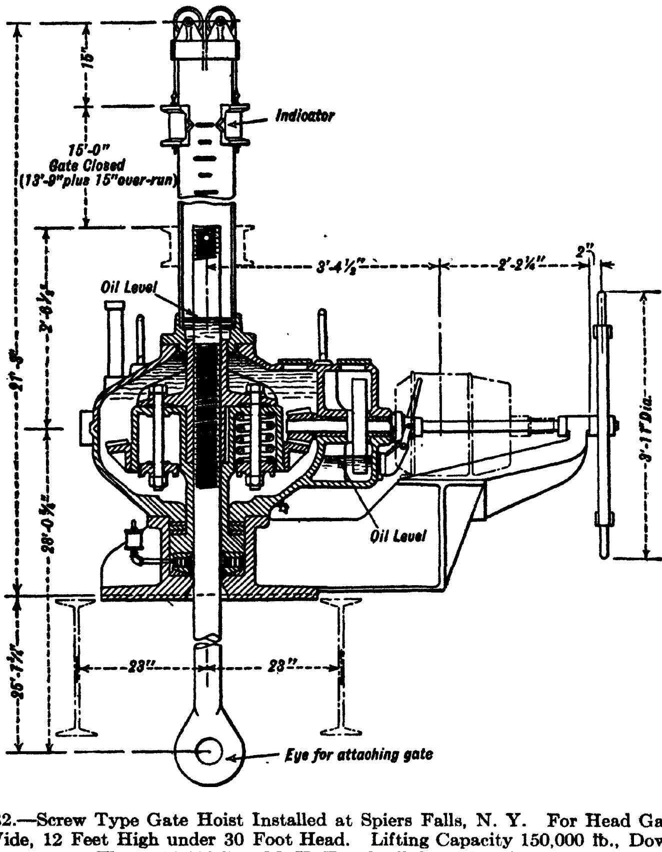


FIG. 232.—Screw Type Gate Hoist Installed at Spiers Falls, N. Y. For Head Gate 18 Feet Wide, 12 Feet High under 30 Foot Head. Lifting Capacity 150,000 lb., Downard Thrust 50,000 lb. M. H. Treadwell Co., Manufacturers.

Large-sized hoists of this type are usually provided with roller bearings under the thrust collar to reduce the friction when lifting. If the force required to close is large, roller bearings are also provided above the thrust collar.

To provide the greatest gear efficiency, the screw should be designed so that the gate will be almost able to overhaul the gearing and start closing under its own weight.

167. Drum Hoists.—Fig. 234 shows a drum type of hoist intended for the operation of a caterpillar gate; but very similar drum hoists are also used for the operation of Stoney and other gates capable of closing under their own weight.

Figure 235 shows a type of hoist particularly adapted to the operation of Taintor gates. The Taintor gate is usually raised by means of two cables or chains attached to the front lower corners of the gate. When the gate is raised the chain or cable winds up in the grooves on the drum shown in the figure. In some Taintor-gate hoists, the drum consists of two truncated cones with the small end of each cone near the ends of the shaft. The chain starts to wind up at the small ends of the cones and thus a greater torque is obtained in starting.

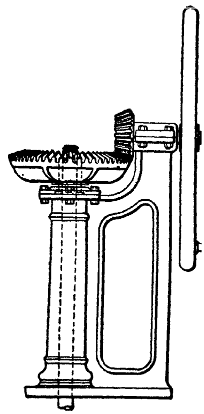


FIG. 233.—Typical Floor Stand for Small Sluice Gate. Ludlow Valve Co.

Drum hoists may be equipped with chains or steel cables. Chains are usually adopted unless multiple sheaves (Fig. 234), are necessary.

168. Hydraulic Hoists.—The hydraulic hoist consists of a cylinder in which a piston, connected to the gate, lifts and lowers the gate by means of hydraulic pressure. The pressure is provided by pumps using oil as a fluid.[13] Pressures as high as 1000 lb. per square inch have been used, but experience has shown that pressures of about 200 lb. are preferable.

The piston packing, particularly under heavy pressures, is seldom tight enough to hold the gate open, and a latch or some other means must be used to support the open gate. For this and other reasons, hydraulic hoists are not particularly well adapted to remote control.

A typical installation of gates operated by hydraulic hoists is at the Vernon Station of the New England Power Company. The following is from a description of these hoists written by Mr. E. A. Dow for the 1925 Spring Meeting of The American Society of Mechanical Engineers.

A fairly typical low-head installation is illustrated in Fig. 236, which shows the gates used for the Vernon extension units put into service in 1921. Each of these units has a capacity of 5000 kw. under 34-ft. head and discharges about 1800 sec.-ft. Each unit has two gates 17 ft. 6 in. high by 15 ft. 9 in. wide, operated by 28-in. oil cylinders.

In line with New England Power policy, these gates are designed to close in an emergency under full head with no back pressure, so that it is possible to shut off the flow with the turbine gates open or with the turbine entirely wrecked. To aid in closing under emergency conditions, the gates were made very heavy (120,000 lb. apiece) the construction consisting of a structural-steel frame filled with concrete. No skin plate was used. The weight of the gates plus the thrust of the hoists is sufficient to insure closure with a coefficient of friction of gate on guides of well over 0.75. They are opened with pressures equalized by means of filler gates. These gates are of the plain sliding type and have seats of structural steel bearing on cast-iron frames.

[13] For a description of such hoists at the turbines of high-head developments, see Sec. 190.

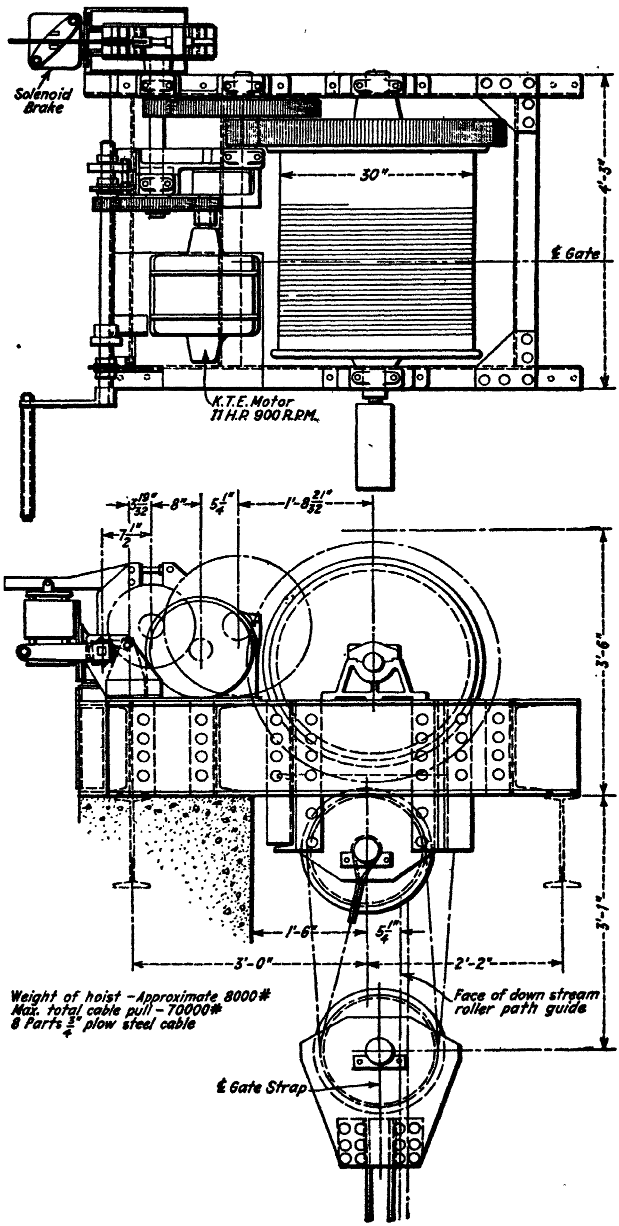


FIG. 234.—Drum Type of Gate Hoist as Made by Philips & Davies, Kenton, Ohio.

Cylinder hoists such as are used for these gates have their own inherent advantages and disadvantages. They are extremely rugged and free from complication and are probably less subject to damage by careless or incompetent handling than any other type. A relief valve prevents the development of excessive pressure and nothing is injured if the gate closes against an obstruction, as the pressure simply builds up until the valve discharges. Under similar conditions a mechanical hoist would ordinarily buckle the stem, strip gear teeth, or otherwise wreck itself. As an offset to these advantages

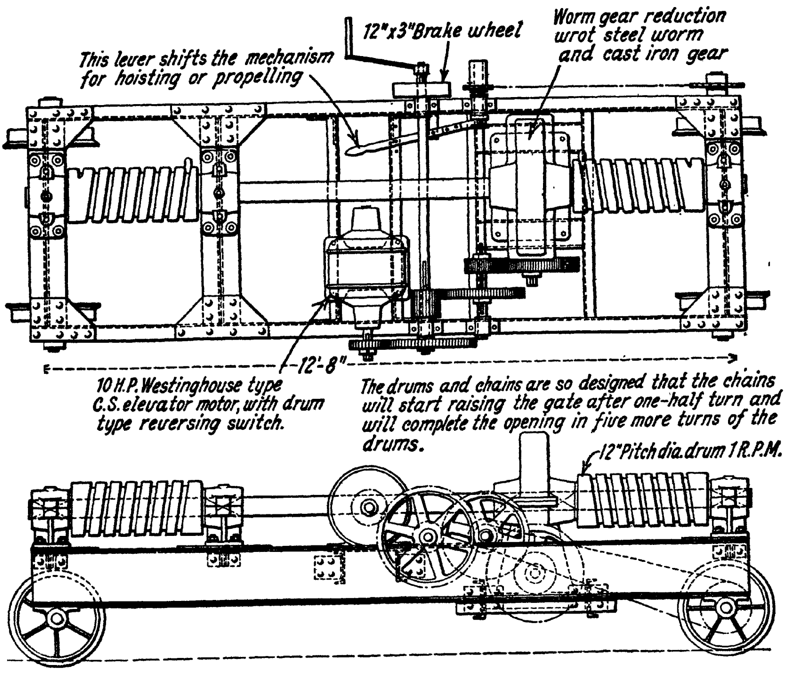


FIG. 235.—Movable Hoist for Taintor Gates, Kerckhoff Dam, San Joaquin Light and Power Corporation.

when used with heavy gates they involve some type of latch to sustain the weight when the gates are raised, and this is a complication if, as sometimes happens, it is desirable to provide for closure by remote control in an emergency. The Vernon gates are supported when open by struts of heavy pipe and it is necessary to raise them and release these latches before closure. If the cylinders are used in a cold climate in an exposed position they must be operated with some liquid which will neither freeze and damage the apparatus nor become so viscous as to refuse to flow. The Vernon cylinders are filled with low cold-test hydraulic-jack oil. Stems must be smooth, otherwise the wear on the stem packing and consequent loss of oil will be excessive. Cylinder walls must be smooth to prevent undue wear of piston packing. Ample pump capacity is necessary so that cylinders can be operated in spite of minor leakage past the piston. The New England Power Company has adopted a minimum pump capacity of 30 gal. per minute for such installations for this

reason and greater capacity is very desirable, even though the desired speed of operation could theoretically be obtained with much less.

The handling of heavy as opposed to light gates by hydraulic cylinders introduces difficulties which most of us only come to appreciate through sad experience. The lowering of gates of this type is ordinarily accomplished by by-passing the operating fluid from the bottom to the top of the cylinder, the weight of the gate pulling on the piston developing the pressure which forces the fluid from the bottom to the top. This means that motion is

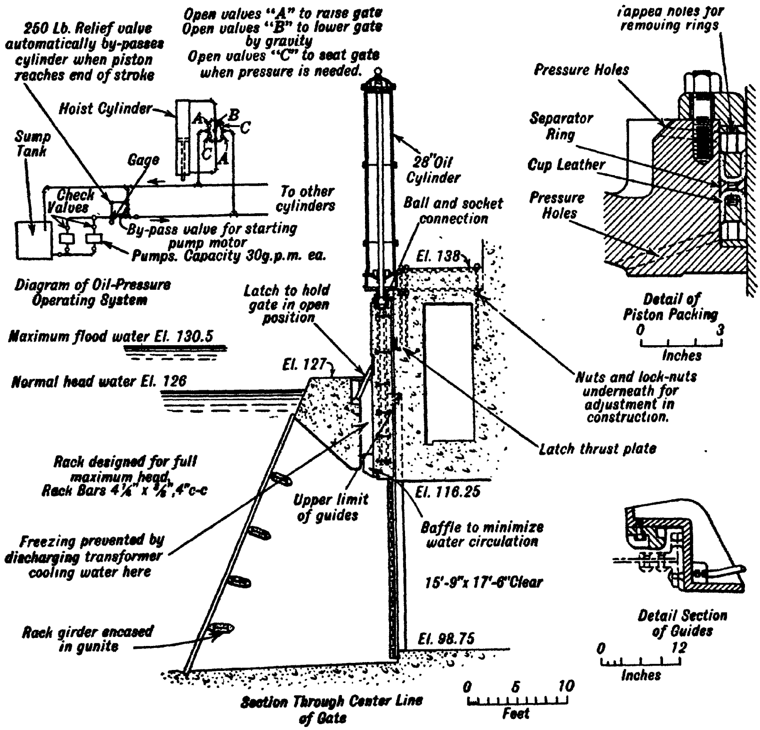


FIG. 236.—Hydraulic Hoists of the Vernon Plant, New England Power Co.

taking place *against* the pressure, and unless the piston leathers are provided with a separator ring of exactly the correct shape there is a tendency for the lower cup or U-leather to be drawn upward past the upper leather. The result is a severe folding and wedging against the cylinder walls, and the leather soon wears through. The author has a vivid recollection of struggling in the wee small hours with a headgate of his design which controlled a 24,000-h.p. plant scheduled to go on the line at 7:00 A.M., and which refused to open because the piston leather had cut through. The gate was only persuaded to operate by feeding the pump enough bran partially to calk the cut and check the leakage. A further peculiarity becomes evident if pressure is admitted to the top of a cylinder while a heavy gate is being lowered or is being sustained by the pressure below the piston. Under this condition the

pressure from the pump is superimposed on that developed by the weight of the gate and the result is quite likely to be a pressure approaching twice that which the designer anticipated. This possibility should be avoided by the proper arrangement of connections or by relief valves. The above may appear to be a serious indictment of hydraulic-cylinder operation, but it is less so than it sounds. Most of the troubles experienced with hydraulic cylinders are vexatious rather than disastrous and are quickly rectified, while a wrecked mechanical hoist is an entirely different matter. Even a doubling up of pressure is unlikely to damage more than the pressure gage, as most cylinders and piping systems have a sufficient factor of safety to stand it. A mechanical hoist equally fool-proof would be extremely difficult and expensive to build.

The Vernon headgates probably meet the requirements of the situation more completely than most similar installations. Men can work behind them without interference from leakage (generous drains take care of what cannot be stopped off by calking) and they can feel confident that the gates will not open of their own accord. The first cost of the installation was not unreasonable and the maintenance and deterioration are practically nothing. Interference with closure due to freezing is entirely eliminated by piping the slightly warm discharge from the water-cooled transformers into the gate wells, which are planked over better to retain the heat. The area is sufficiently large so that the head loss is practically eliminated.

169. Motive Power.—Gate hoists may be readily operated by hand, by electric motor, or by hydraulic pressure.

Trautwine states that an ordinary man is capable of exerting a 16-lb. force on a hand crank, at a speed of 150 ft. per minute, for eight hours with a 25 per cent allowance for rest. The crank effort required in starting the gate usually becomes rapidly less as the gate is lifted. The crank effort in starting should not exceed 40 lb. for gates normally closed and 30 lb. for gates normally open. These values will correspond to about 20 lb. for the effort after the gate is started.

The most convenient radius of the hand crank is about 15 in. with shaft about 3 ft. above the floor. The radius of hand wheels varies considerably among manufacturers. The advantage of a hand wheel is that it can be spun rapidly under light loads. At present, however, hand wheels are seldom used and, in most instances, provision is made for changing the crank to a high-speed shaft when the load is light.

When the hoist is of such large capacity that hand operation becomes tedious, or when remote control becomes necessary, an electric motor is provided. However, the hoist should always be provided with emergency hand operation for use if the motor control should fail.

Alternating or direct-current motors may be used for gate hoists, the choice depending upon the kind of current adopted for the other station auxiliaries. Ordinarily, variable-speed, high-starting-torque motors are required as there are very few cases where the type of hoist and gate is such that the motor can attain full speed before being loaded. Constant-speed motors have been used successfully with drum hoists where the slack in the chain or cable, when the gate is closed, is sufficient to allow the motor to attain speed before the gate starts to lift; but this arrangement limits the adaptability of the motor. As an example, if such a gate were opened only slightly in order to fill the

conduit gradually, it could not be opened further without first closing it to get slack in the cable to permit the motor to start.

Direct-current motors have been more frequently used in the past for station auxiliaries, but alternating-current auxiliaries are now coming into greater favor. Alternating current is not as well adapted to variable-speed motors as direct current; but it is suitable for the operation of gate hoists.

Variable-speed d.c. motors are series-wound. Variable-speed a.c. motors may be of either the high-resistance, squirrel-cage type or the slip-ring type.

Successful operation of gate hoists by motors requires accurate control at the end of gate travel, particularly for cases where overtravel beyond the desired point will jam the gears and cause damage. Almost every type of gate provides ample leeway at the end of travel when opening; but, except for drum hoists, which can exert no direct downward thrust, the permissible overtravel at the end of the closing operation is very small.

The type of gate sill shown in Sketch *C* of Fig. 200 is best adapted to motor operation, as the gate is closed several inches above the point of maximum possible travel. This is not the case for the sills shown in the corresponding Sketches *A* and *B*.

The gate has a tendency to travel a short distance after the current is cut off the motor, because of the inertia of the moving parts. Therefore, when the permissible overtravel is limited, the speed of operation at the end of gate travel should be reduced, to facilitate stopping at the proper time. A reduction of speed at the end of gate travel is accomplished by the use of hand controllers. Such controllers are difficult to operate from a distance and hence, for remotely controlled hoists, considerable permissible overtravel should be provided and the controller for reducing speeds should not be used. Controllers are usually furnished with a number of speeds, forward and back, although in many cases the danger from overtravel may be only in one direction. Controllers are not required for closing with a.c. squirrel-cage motors or d.c. series motors if, as in the case of sliding gates, the load is greatest at the end of gate travel, because, for these types of motors, the speed is slowest when the load is greatest. Slip-ring a.c. motors, on the other hand, are variable in speed only when starting, and controllers for stopping under these conditions would be required.

Limit switches should be provided for all installations. Such switches are essential for remotely controlled hoists, to throw off the current and stop the motor at the proper time. In the case of hand control, they are very desirable to prevent overtravel due to carelessness of the operator. They should be adjusted to the inertia of the moving parts so that the continuance of gate travel after the current is off will not reach the limit of permissible travel and jam the gears.

Solenoid brakes should be provided for those gates which may overhaul the gears and close of their own weight. Such brakes are designed to set automatically when the current is cut off the motor. They are also useful to reduce the length of travel due to inertia, as previously explained.

The National Electric Light Association recommends [14] the use, in addition

[14] See Report of Hydraulic Power Committee, 1924.

to a limit switch, of a friction drive consisting of a fabric disk held by springs between metal disks. This type of drive is most useful where there is danger of an obstruction lodging under the gate and causing jamming before the limit switch stops the hoist. However, the danger from such obstructions, at intakes protected by racks, is not great, and a friction drive adds one more part to maintain.

Overhead trolleys are considered best for motor-operated traveling hoists. Plug-and-receptacle systems have been used but are frequently a source of trouble, particularly where the removable connecting cable may be run over when moving the hoist.

An electric motor may be provided with each hoist; or, by means of a line shaft with clutches, a single motor may be used to drive successively two or more hoists. A motor with each hoist is most frequently used.

170. Traveling Hoists.—It is a common practice to mount drum hoists on wheels, to travel along the top of the intake and operate each of the headgates. Such a hoist is shown in Fig. 235.

There are some objections to traveling hoists in all but low-head developments, as in some conceivable emergencies it might be very important indeed that the headgate controlling some particular unit or penstock be put down as promptly as possible. If the hoist were far removed from the gate affected, it would take quite a while to unhook it from the gate to which it was then connected, move it to the gate to be operated, attach it, and lower the gate. By the time the hoist was in position, untold damage might be done, which might have been avoided had the gate been hooked up to a hoist ready for instant operation.

To overcome this objection, latches have been arranged at some installations, so that in an emergency, the headgates could be dropped into position.

In any particular case, the engineer charged with the responsibility for the project must balance the saving in first cost effected by using one hoist for several gates, against the assurance which an individual hoist for each gate gives. For moderate and high-head developments of the first magnitude, it is believed that in most cases it will prove worth while to install an individual hoist for each gate.

Traveling hoists are usually mounted on a simple car or truck provided with a track. However, in some instances, where the intake has been provided with a superstructure, a power-house type of crane has been provided and used not only to operate the gates but to handle trash racks, raking devices, and stop-logs as well. In the absence of a superstructure, a gantry crane has been used for this purpose.

171. Intake Structures.—As previously explained, intakes are divided into two main types; but the types have no clearly defined line of demarcation.

(a) *Low-pressure intakes*, which correspond to only a few feet drawdown of the pond, as in Fig. 237;

(b) *High-pressure intakes*, which correspond to a very considerable drawdown, as in Fig. 241.

The essential requirements of the two types are the same; but the details

and type of equipment may be radically different on account of the difference in the depth of water in which they must be operated.

When high-pressure intakes are used in conjunction with an earth or rock dam, the structure usually takes the form of an isolated tower, and the details of the structure are radically different from those of a low-pressure intake.

It is customary, in low-pressure intakes, where there is considerable danger of complete and sudden plugging of the racks with ice or trash, to design all portions of the intake for complete stoppage of flow through the racks. In such cases the corresponding hydrostatic pressure on the racks, walls, and other portions of the structure is known so exactly and occurs so seldom that a relatively high unit stress may be used in the design. Reasonable working stresses are 25,000 lb. per square inch in the structural steel, and 33 per cent more than the usual working stresses in reinforced concrete.

For low-head developments in warm climates, if the run of trash is very small, a maximum drop of water surface at the racks of 5 or 6 ft. is a reasonable assumption, provided that the intake is not too isolated and the importance of the installation is such that only first-class, experienced operators are in charge. Under such circumstances, a greater drop at the racks would be the result of many weeks of neglect and would always be observed and corrected to prevent the corresponding loss of power.

As the depth of racks increases, the danger of complete stoppage of the flow becomes more remote. Thus, for high-pressure intakes at the outlet of a large reservoir, the amount of trash reaching the intake may be negligible, and the water is drawn out of the reservoir so slowly that only the part of the racks near water surface could become plugged with ice. Therefore, only the lower portion of the structure need be designed for full stoppage when water surface is low, the reservoir small, and the danger from ice and trash correspondingly greater.

The strength of the walls, buttresses, and other parts of the intake structure should be designed for all possible conditions of operation and unwatering for repairs. Should the intake be divided into bays, each commanding a separate unit, as in Fig. 237, it is possible for only one unit to be running, the racks of that unit to become plugged, and the walls between that unit and the adjacent ones to be subject to full water pressure. Each unit should be investigated for stresses due to water pressure when either all or only a part of it is unwatered by the closure of gates or by the installation of stop-logs.

Further details of the two types of intakes are described in succeeding sections.

172. Low-pressure Intakes.—Details of low-pressure intakes vary greatly according to the requirements and the preference of the designer. Typical examples are shown in Figs. 237, 238 and 239, which follow, and also in Figs. 204, 213, 214 and 216.

The raking platform should be placed as close to water surface as possible and, for this reason, it is frequently only slightly above ordinary high-water surface, with the main wall carried well above maximum flood elevation.

Low-pressure intakes frequently perform the functions of a dam and should be designed in accordance with the theory of the stability of dams.

Full uplift must be considered on the top of the conduit below the gates, unless the top and bottom of the conduit are tied together with steel reinforcement, as they frequently are.

Usually a stop-log groove is located close to the upstream side of the gates to provide for unwatering the gates for inspection and repairs. It is usually considered that the necessity for unwatering the rack supports is very remote,

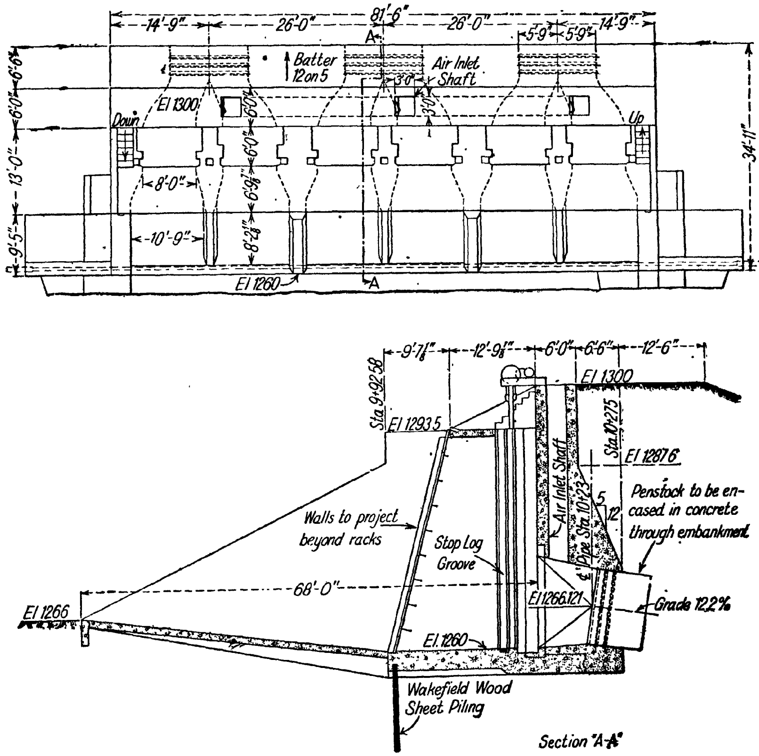


FIG. 237.—Penstock Intake, Soft Maple Development, Northern New York Utilities, Inc.

and only in isolated cases (Fig. 238) are stop-log grooves provided upstream from the racks. However, the walls between the bays are frequently allowed to project beyond the racks, as in Fig. 237, to provide a seat for an inclined bulkhead in case such unwatering should become necessary.

If the seats for the gates are embedded in the main portion of the concrete, it is very difficult to set them accurately. When accurate alinement of the seats is required, recesses are frequently left in the concrete and the seats are grouted in after the gate is set in position and the gate and seats held in contact to accurate position.

Air inlets are necessary in all intakes for closed conduits, as explained in Sec. 182. These are provided by a vertical shaft in the concrete below the gates. Such shafts should be equipped with ladders to provide access to the conduit at that point. Special requirements of air inlets at intakes, particularly as regards the prevention of freezing, are explained in Sec. 182.

The desirability of a superstructure or housing on the intake depends upon climatic and other conditions and upon the type of gate hoist provided. In very cold climates it is advisable to provide a superstructure to protect the rack tenders if much raking is necessary. The superstructure frequently extends out over the racks, and a curtain wall extending to water surface is

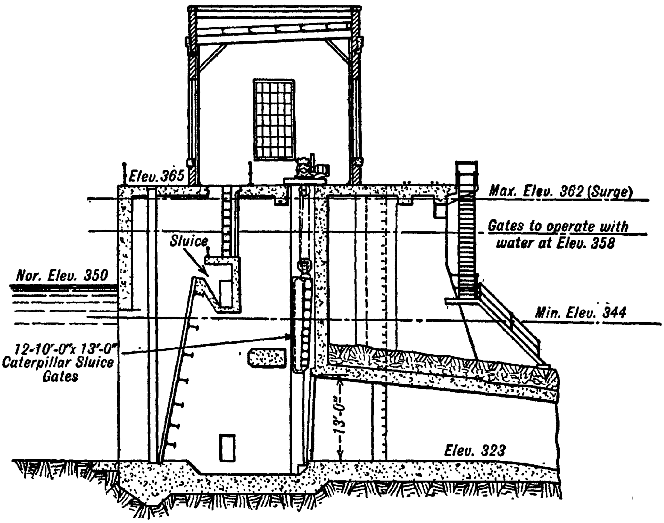


FIG. 238.—Intake for Sherman Island Development, International Paper Company.

provided, as shown in Fig. 238. This arrangement seals the interior from the outside air.

If the racks are allowed to project above water surface and the air within the superstructure is kept warm, the transmission of heat down the rack bars may reduce the amount of anchor ice which will cling to the racks.

A heated superstructure will prevent the formation of surface ice at the gates. In many cases this ice has accumulated to such an extent as to delay the closure of the gates.

In cases where frazil and anchor ice troubles are frequent, and there is little or no trash in the water, it is customary to provide for removing the top section of the racks when anchor ice is running. Anchor ice usually runs during periods of low water when the least amount of trash is present. As the racks become plugged very soon after anchor ice appears, it is necessary to remove them very quickly. As they ordinarily weigh from 15 to 20 lb. per

square foot, even a small section is difficult to remove quickly by hand. In some instances, where a superstructure has been provided, an ordinary power-house traveling crane has been used to handle both the gates and the racks. In the absence of a superstructure, a gantry crane has been used. Frequently, however, where the pond is small and shallow and the turbines small, the presence of trash will not permit the removal of the racks.

When much trash and cake ice must be taken care of, it is of material assistance to the rack tenders to provide a sluice over the trash racks to carry it away. Such an arrangement is shown in Figs. 238 and 239 and consists of a sluice, one side of which is a weir at the top of the racks. The water may be allowed to pass over this and carry with it the floating ice and trash, as well as the ice and trash removed from the racks. The sluice discharges below the dam. The elevation of the crest of the weir at the Deferiet Intake may be adjusted by means of stop-logs in order to regulate the depth of overflow.

During periods of abundant river discharge the water is allowed to pass over the weir continuously; but, as the capacity of the sluice is limited, the crest of the weir must be adjusted to the elevation of head-water surface in order to prevent choking of the sluice with water. In order that operation may be most effective, there must be a decided drop at the weir, and the discharge must be limited to that which will not cause back water in the sluice to submerge the weir.

During periods of low river discharge, when there is little trash or ice to dispose of, the stop-logs on the weir are extended above head-water surface, thus stopping the flow over the weir. The trash is then raked over the stop-logs

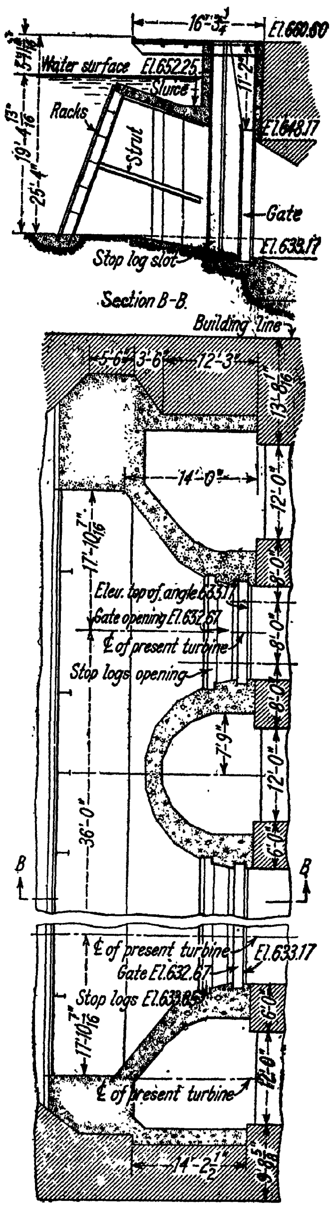


FIG. 239.—Deferiet Intake of the Northern New York Utilities, Inc.

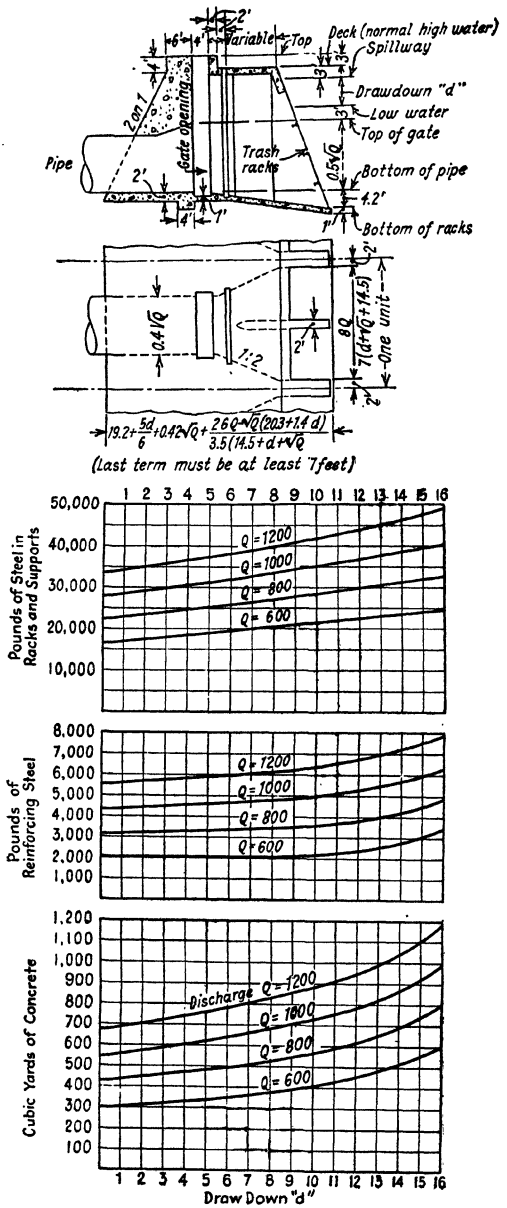


FIG. 240.—Quantities in One Unit of Intake.
See text for description, assumptions and limitations.

into the sluice and flushed out at intervals by opening a gate at the upper end of the sluice.

The sluice should be made as deep and wide as the type of intake will allow. Considerable width is particularly required if large blocks of cake ice are to be disposed of. The raking platform should be cantilevered out as indicated in the illustrations, to avoid the necessity of other supports which would interfere with cake ice passing into the sluice. The stop-logs at the Deferiet weir are supported between removable I-beams resting on the permanent weir crest and supported at the top against the raking platform.

Figure 237 shows a very desirable feature at the outer edge of the raking platform, particularly for exposed conditions in cold climates. This detail provides for a sloping up of the platform, normal to the pull of the rakes, and reduces danger from slipping.

In high-head developments the cost of the intake is frequently a very small portion of the total cost of the project. In such cases preliminary estimates are frequently made without the aid of a detailed

design of the intake. For such purposes Fig. 240 has been prepared for this book by Mr. F. H. Burnette. It gives approximate quantities of materials in one type of intake for various discharge capacities and drawdown of pond. The designs upon which the quantities of Fig. 240 are based are as follows:

Elevation of top of intake	6 ft. above crest of dam
Quantity of water per unit	Q
Velocity in pipe	8 ft. per second
Velocity at racks (Average draw-down)	1.75 ft. per second *
Velocity at racks (Maximum draw-down)	2.00 ft. per second *

* Whichever requires the larger area of racks.

The quantities shown in Fig. 240 are for one unit of the intake. For N units, multiply the quantities by N and then add the following:

For concrete: Two feet on each end to make end walls 4 ft. thick.

For reinforcing steel: One-quarter of that given for one unit.

For racks and supports: No addition.

All concrete for necessary wing walls, and for foundations below that indicated, should also be added. Quantities should be altered for differences between assumptions and actual conditions, particular attention being paid to the elevation of top of intake above crest of dam, which may not correspond to actual flood-water elevation.

Quantities of steel in racks and supports are based on full stoppage of flow by rack plugging and a stress of 20,000 lb. per square inch under such conditions. The racks are 3 by $\frac{3}{8}$ in., spaced 3 in. centers.

173. High-pressure Intakes.—High-pressure intakes may be built in a variety of forms. If the dam is of concrete, the details of the intake differ very little from those of the low-pressure intake described in Sec. 172, except in the type of gates. If the dam is an earth or rock-fill structure, the intake is usually in the form of a tower located near the foot of the upstream slope, although special forms of intakes, located in shafts of outlet tunnels, have been used in a few cases. A typical modern high-pressure tower intake is shown in Fig. 241.

Tower intakes should be sufficiently stable to resist ice thrust, wind pressure when the pond is empty, and, in some sections, the shock of earthquakes.

It is unfortunate that no accurate data are available on the probable thrust of ice. A general discussion of ice thrust on dams is given in "Engineering for Masonry Dams";[15] but it is very improbable that many existing tower intakes could successfully withstand ice thrust of the magnitude that has been used in the design of some dams. It is therefore necessary to compare the proposed design with the dimensions of existing intakes of about the same height under similar ice conditions.

The following discussion of the force generated by earthquakes is taken from the 1924 Report of the National Electric Light Association, Hydraulic Power Committee.

[15] By W. P. Creager, John Wiley and Sons.

In regions subject to earthquakes it is advisable to design the tower for a horizontal loading due to the acceleration of the quake rather than for the customary wind force of 20 lb. per square foot on the projected area of the tower. The horizontal force due to the earthquake is as follows:

$$\text{Earthquake force} = \text{mass} \times \text{acceleration} = \frac{\text{weight}}{G} \times \text{acceleration.*}$$

Professor A. C. Lawson, following the San Francisco earthquake of 1906, estimated the accelerations which had occurred on various formations as follows:

Foundation Material	Acceleration ft. per sec.^{2}
Serpentine	0.8
Made land	3.6
Marsh	9.8
Sandstone	0.8 to 2.0
Made land	9.5
Sand	2.0
Sandstone	1.3
Sand (Mission Valley)	3.6
Marsh	9.8
Sundry solid rocks	0.8

It has become common practice on the Pacific Coast to design for an earthquake acceleration of 6 ft. per second per second where the tower has solid foundation. This force is placed at the center of gravity of the portion of the tower above the section considered. Should the tower be built on other than solid rock it may be wise to increase the acceleration given above to a higher figure, since records show that the accelerations in soft material are much larger than in firm unyielding rock. The acceleration, of course, may be reduced proportionately as the distance the tower is located from the fault line increases. The most probable point of rupture in a monolithic tower is at or near the center of percussion, though this is not universally true. In towers over 110 ft. in height, if earthquake shocks are designed against, as outlined above, it will usually be found that stresses caused by wind need not be considered, as its effect will be much less (usually not more than 50 per cent) than that of the earthquake.

Where earthquake stresses are considered it is common practice to increase the allowable stresses on the concrete and steel from 25 per cent to 50 per cent.

The Davis Bridge Intake, shown in Fig. 241, has two gates located in the bottom of the tower, enclosed in a cast-iron conduit which excludes the water from the interior of the tower. Leakage, in towers of this type, should be drained to a point below the dam. There should always be two gates and provision should be made that one may be bulkheaded off for inspection or repairs while the flow for the turbines is being passed through the other. This feature was incorporated in the Davis Bridge Intake.

A bridge is frequently installed between the dam and the tower, to permit easy access and the replacement of apparatus. Housing on top of the tower is usually provided in cold climates. It is advisable, if the apparatus in the intake is heavy, to provide supports for a chain block or other means of lifting the apparatus out of the interior of the tower.

* EDITOR'S NOTE.—G, as here used, is the acceleration of gravity and is equal to 32.2 in the foot-pound-second system of units.

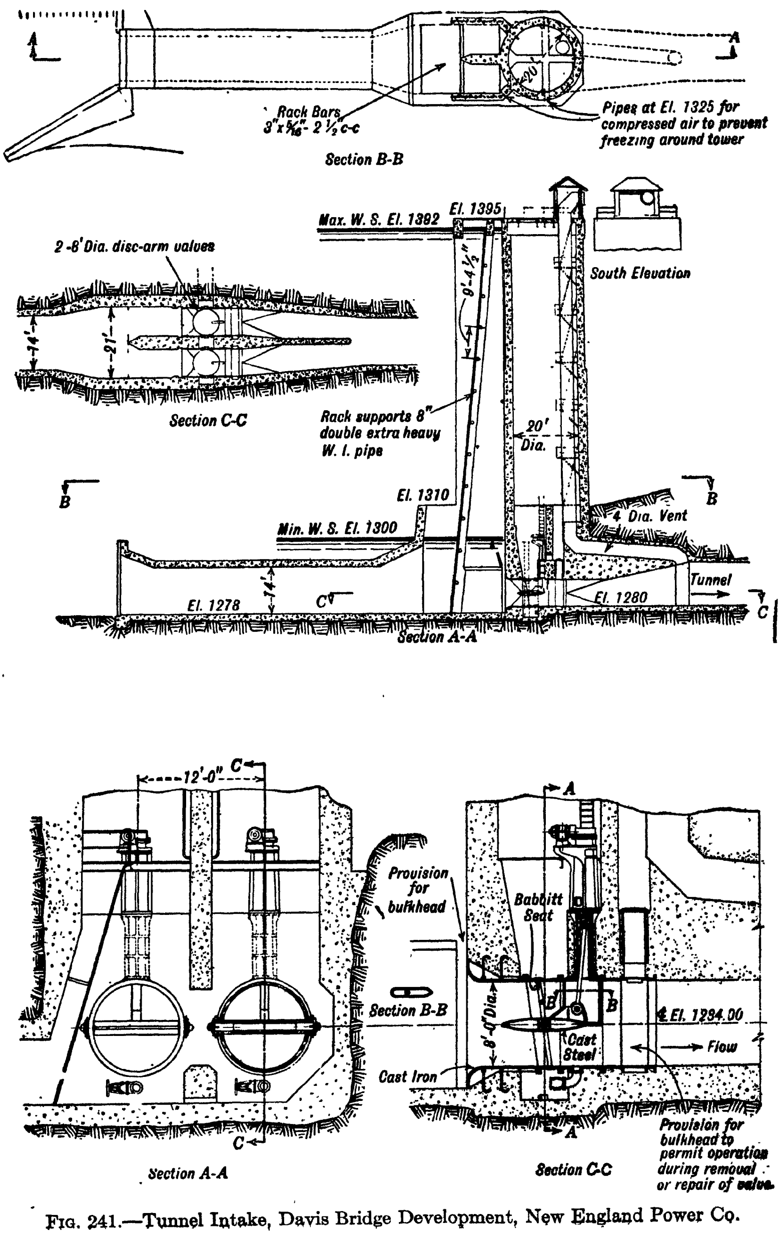


FIG. 241.—Tunnel Intake, Davis Bridge Development, New England Power Co.

Much of the discussion of details of low-pressure intakes, given in the preceding section, applies equally to high-pressure intake design.

174. Trash Racks.—Trash racks are made of steel bars of rectangular

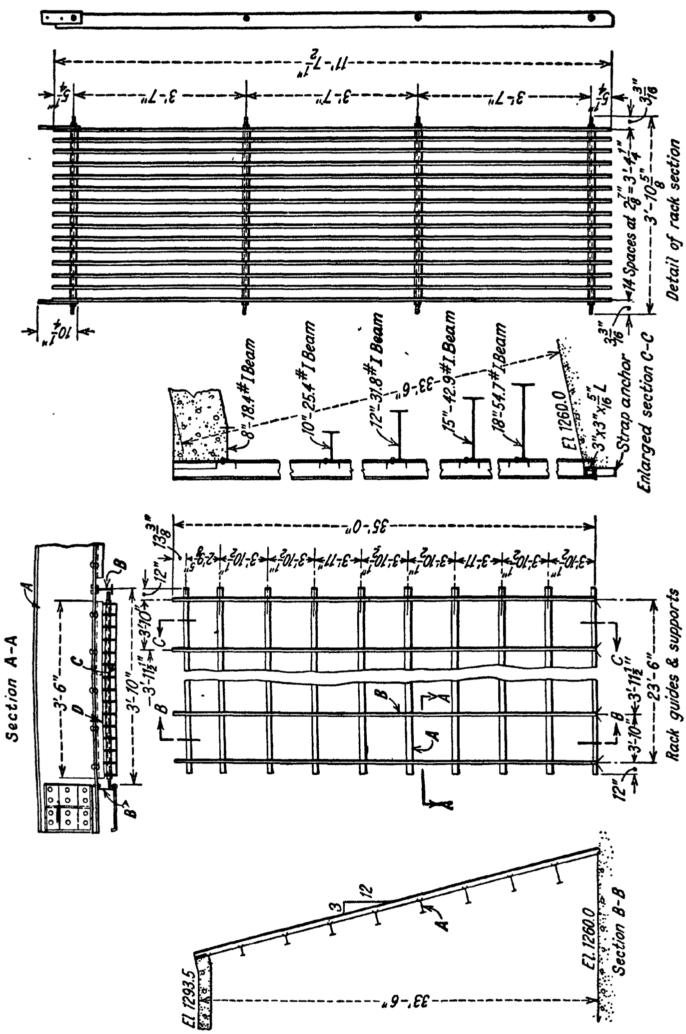


FIG. 242.—Trash Racks at Soft Maple Development, Northern New York Utilities, Inc.

cross-section set parallel to each other. They are usually built in panels 4 or 5 ft. wide and of a length not too great for handling. Fig. 242 shows details of the racks of the Soft Maple Development, the intake of which is shown in Fig. 237. The main supports for the racks consist of horizontal

I-beams, *A*, spanning each intake bay. Attached to the supports are the inclined I-beam and channel guides, *B*. The rack spacing bolts, *C*, project sufficiently to slide within the inclined guides. The racks rest on the small channels, *D*, attached to the main supports, which serve simply as a filler to make the racks project beyond the outer flange of the inclined guides.

The rack bars are usually $\frac{1}{4}$ or $\frac{3}{8}$ in. thick, the latter thickness being more frequently adopted on account of the desirability of greater stiffness in handling. They are usually $2\frac{1}{2}$ to 3 in. wide. In the case of very wide spacing of bars for large high-speed units, the bars may have to be made larger in order to be structurally safe in case the racks become completely clogged. At intervals, the bars are drilled as close to one edge as practicable, and rods $\frac{5}{8}$ or $\frac{3}{4}$ in. in diameter are passed through them. Thimbles, obtained from short lengths of steel pipe, are threaded on the rods to act as spacers to keep the bars apart. The rods are threaded at the ends, and a section of the bars is bolted together. The edge to which the drilled holes are closest is toward the downstream side of the racks, so that the teeth of the rake may pass between the rack bars without interference and remove anything that is lodged on or between them.

The maximum clear distance between the rack bars is usually dictated by the turbine manufacturer and is fixed by the smallest clearances through the turbine. It usually varies from 2 to 4 in. for ordinary units in medium-head plants. Closer spacing is required for very small units, and particularly for impulse wheels, while for the largest sizes of high-speed runners a spacing as great as 6 in. has been used.

If no crane is provided for handling the racks, they are usually made in sections sufficiently light for hand removal and replacement, and are built up before being lowered into place. However, where the top sections are to be removed during times of anchor ice and a crane is provided for lifting them, they are sometimes made in very large sections as in Fig. 243.

It is essential that the sections be stiff enough to prevent springing during handling. This feature is particularly necessary if the top sections are to be removed when anchor ice is running, as previously discussed.

No projection of any kind, which will engage the rake teeth and interfere with raking, should be allowed on the racks or the supports. If the racks are made in sections, the sections must join perfectly so that each bar of one section will line up with the corresponding bar of the one below.

Steep racks are difficult to rake because, for effective raking, there should be a component of the weight of the rakes to hold the rakes against the racks. Opinions of designers regarding the proper inclination of the racks vary widely, and intakes with inclinations varying from 45 degrees to vertical may be cited. However, this variation is not pronounced in the more modern installations but ranges about as follows: for low-pressure intakes where the run of trash is great, an inclination of 18 to 20 degrees with the vertical; where the run of trash is light as at the outlet of a large pond, an inclination of 14 to 16 degrees. In high-pressure intakes there is usually little necessity for raking, and racks that varied far from being vertical would be very expensive if the intake were high. Consequently, for high-pressure intakes, the racks are made vertical or nearly so.

The rack supports are frequently designed not only as beams to support the racks but also as struts to support the side walls against water pressure.

The possible loading of racks and supports, due to ice and trash accumulations, and the allowed stresses under loaded conditions are explained in Sec. 171.

Approximate weights of racks and supports for the usual type of low-pressure intakes are given in Fig. 240.

The usual velocities of flow through rocks are discussed in Sec. 151.

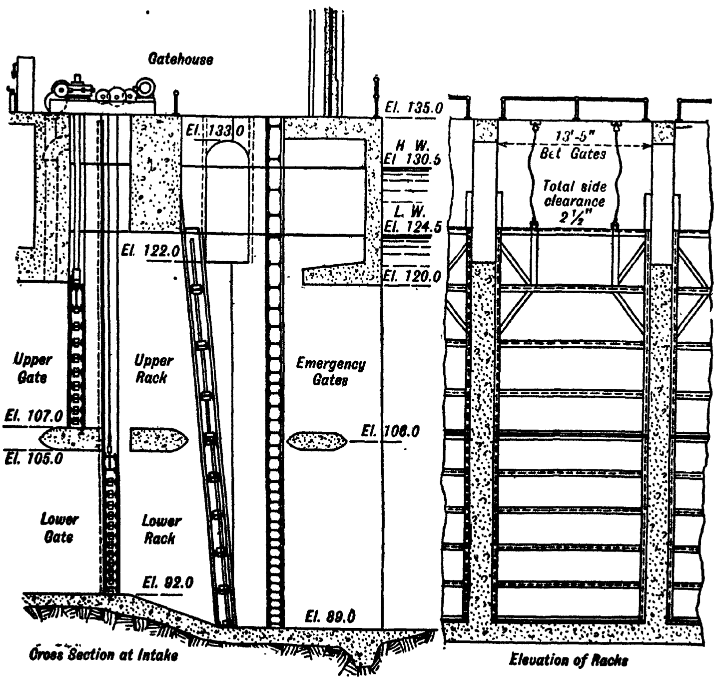


FIG. 243.—General Arrangement of Gates and Racks. Cedars Rapids Manufacturing Power Company, Cedars, Quebec.

175. Raking Racks.—At most plants, hand raking of racks is depended on. On most streams it is necessary to do very little raking of racks during the greater part of the year, but in the fall of the year, when the streams carry a great many leaves, intensive raking is frequently required. Also, at times when the streams are carrying a large amount of débris, raking becomes quite a problem. Streams vary greatly in this respect; on some, in order to keep the racks sufficiently free of leaves and débris to permit the efficient operation of the units, constant raking with a considerable force of men is required at certain seasons of the year.

The rake made up for use in hand raking consists of iron or wooden pins so spaced that they will fit in between the rack bars. The handle of the rake sometimes consists of a steel pipe, but this makes a rather heavy rake for deep raking. A convenient rake consists of a rake-head of iron with teeth to cover a width of rack bars of about 18 in., and a handle of selected ash about 1½ in. in diameter.

Most operators consider that 20 ft. is about the limit of hand raking, although it is possible, with considerable difficulty, to reach still deeper if the rake-handle is disjointed as it comes up. In many cases most of the trash

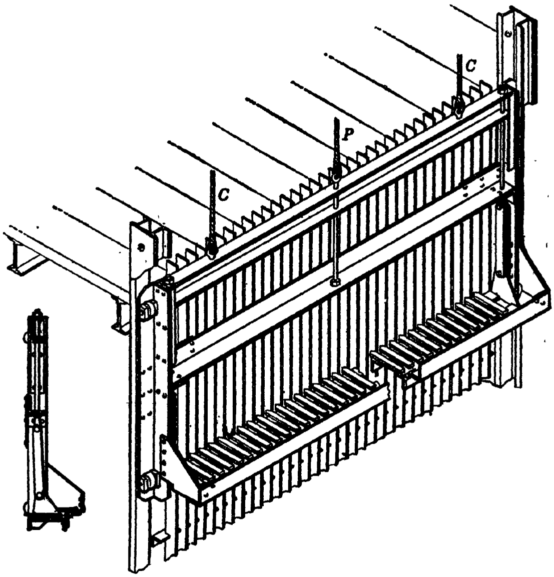


Fig. 244.—Mechancial Rake. Newport News Shipbuilding and Drydock Co.

is near the water surface; but this is decidedly not the case in streams where turbulent water is close to the intake.

In cases where a great deal of raking is required, it frequently pays to install a rack-raking machine. There are now a number of these machines in use at various plants, and they are generally proving effective in keeping the racks free of débris under severe conditions. They vary from crude rakes of large size which are lowered by cable from a traveling winch and guided by hand, to most elaborate devices which are entirely mechanical in their operation. The wholly mechanical type of rake seems to be preferable, if such devices are to be used at all. Such a machine, as made by the Newport News Ship and Dry Dock Company, is shown in Fig. 244. In this figure, when the rake is being lowered from a drum hoist, tension is on the line P, and the teeth of the rake hang loosely with the back ends lower than the front. On the

ascent, the tension is on the line C, which pulls the teeth up straight and causes the front end of the teeth to ride in between the racks and do the raking.

Another raking device is shown in Fig. 245. This rake travels along the intake and rakes one section of the rack at a time. By a suitable mechanism on the hoist, the rake is kept parallel to the racks when lowering, and normal

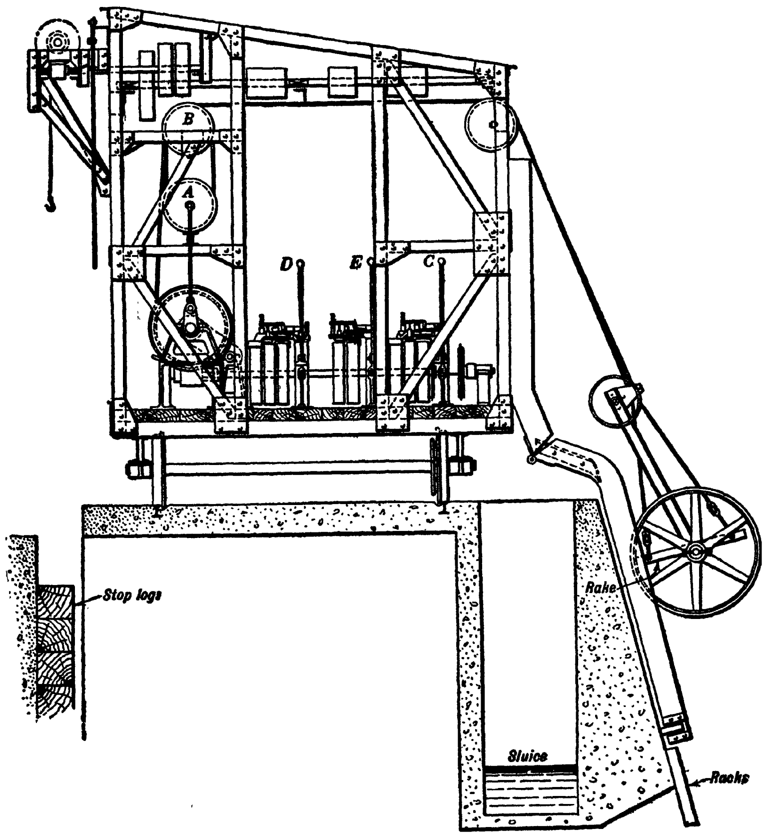


FIG. 245.—Mechanical Rack Developed by J. W. Jones.
(Power,. Jan. 24, 1922.)

to the racks and in a position for raking when lifting. A full description of the rake is given by Mr. I. W. Jones in the Jan. 24, 1922, issue of *Power*.

Figure 246, furnished by Mr. O. G. Thurlow, Chief Eng., Alabama Power Company, shows details of a compressed-air device which has been successfully used for removing trash from the Mitchel Dam racks. In this device, air is discharged at the foot of the racks and, traveling upward along the racks, carries the débris to the surface.

The figure shows only one of the three bays so equipped. The piping throughout is $1\frac{1}{4}$ in. iron pipe; but it is proposed to use brass pipe for greater permanency in the Cherokee Bluffs Development, which is similarly equipped. Details of the nozzle pipe, P, at the base of the racks are shown in the figure. In this pipe are drilled $\frac{3}{16}$ in. staggered air-outlet holes about $4\frac{3}{8}$ in. apart.

According to Mr. Thurlow, the usual method of operation is to cut the output of the turbine down to one-third of its capacity or to about 0.5 ft. per second velocity through the gross area of the racks, and to turn on 100 lb. of air pressure for about five minutes. This is sufficient to agitate the water and carry trash to the surface. A compressor having a capacity of 250 cu. ft. at 120 lb. pressure is used. There is a 48-in. diameter by 144-in. high receiver on the line. However, both the compressor and the receiver were installed to suit other conditions in the plant

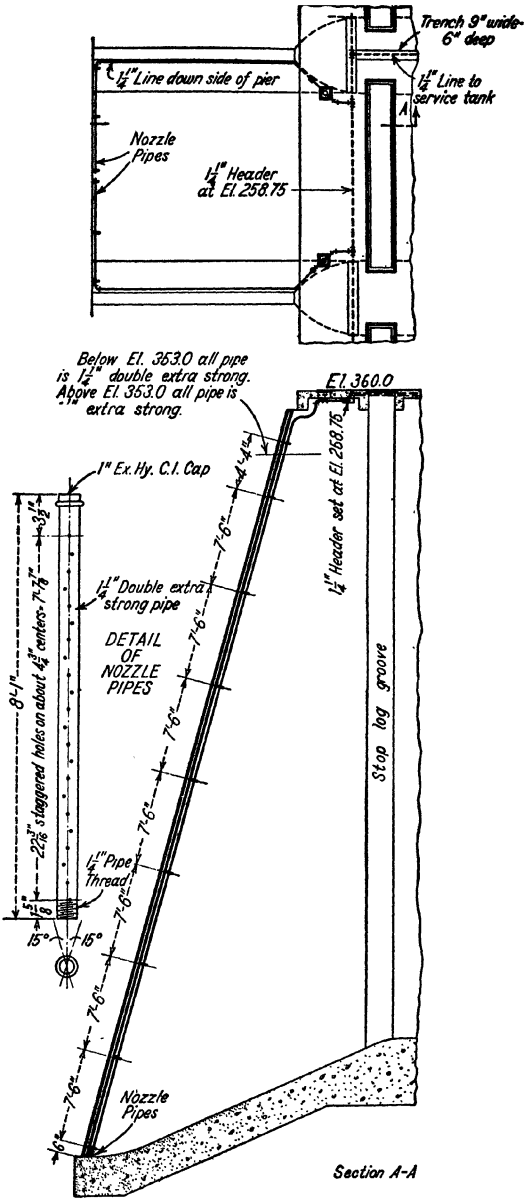


FIG. 246.—Air Rack Raking Device, Mitchell Dam, Alabama Power Co.

and are probably larger than required for air raking. One bay is raked at a time.

For removing trash from the racks, the nozzles should be placed at the foot of the racks, and the velocity of the water during raking operations should be such that the resultant of the vertical velocity of the air and the horizontal velocity of the water has an inclination equal to that of the racks. The air will then follow the racks to the surface.

176. Provisions against Ice Troubles at Intakes.—In the design of intakes located in cold climates, it is often necessary to adopt special precautions to prevent ice from interfering with the operation of the plant.

The operator has to contend with three distinct forms of ice; namely, sheet ice, frazil ice, and anchor ice.

The small ice crystals, which form when the temperature of the water at the surface falls to the freezing point, accumulate to form the ice sheet if the velocity of the water does not carry them away. In swiftly moving water, the ice sheet forms at the sides, where the velocity is relatively low, and gradually builds out until the whole surface is covered. If the velocity is too high, or if fluctuations of the water surface loosen the sheet at the sides as fast as it forms, the surface will not become covered, even in the coldest climates.

When the ice sheet melts, it breaks up into cakes of large size, which float down to the intake and must be disposed of through ice chutes or over the dam. Ice chutes at the lower end of canals are essential to get rid of floating ice.

The small ice crystals, which form at the surface or in rapids, and which are not held back in the formation of an ice sheet, appear at the intake and are known as " needle " or " frazil " ice. In the eddying of the current the crystals clot together loosely in soft masses termed " slush " ice, which float at various depths. This ice has a tendency to adhere tightly to all substances having a temperature equal to or less than that at which water freezes. The adherence of the ice to the racks and the turbines may completely plug all openings within a short time after the ice starts to run. Rakes will not effectively remove this ice from the racks.

" Anchor " ice takes the form of small ice needles which form on the bottom of fairly quiet, shallow bodies of open water or water covered by a thin transparent ice sheet. It forms usually on cold, clear nights, and becomes loosened and floats away during the early hours of the day. In action and appearance it is very similar to frazil ice, previously described.

Except in rare cases, frazil and anchor ice form only when there is no ice cover in the river, pond, or canal. Consequently, when the power house is located in the dam and there is a pond of considerable size above the dam, there is seldom serious trouble from frazil or anchor ice. Similarly, little or no trouble need be expected when the approach to the power-house intake is through a long, deep canal at a velocity low enough to permit the ice sheet to form readily. However, at practically all plants in cold climates, there will be some formation of frazil and anchor ice during the late fall or early winter before the water has a chance to freeze over.

On the other hand, when conditions are such that the water is brought to

or near the intake at a relatively high velocity, which tends to prevent the formation of an ice sheet, a great deal of frazil and anchor ice is frequently formed.

The greatest source of trouble is frazil and anchor ice. Fortunately, these formations are very loosely held together and possess little or none of the structural strength of sheet ice. Accordingly, if they can be prevented from sticking to the surfaces of the racks and turbines and thus causing clogging, they may be passed through the plant without causing trouble.

It has already been stated that frazil and anchor ice will adhere to surfaces that are at or below the freezing temperature of water. If such surfaces can be kept only a very small fraction of a degree warmer than freezing temperature, no difficulty will be experienced. The greatest trouble occurs at the racks, because the turbine parts are usually kept at a temperature above freezing by conduction from the warm air surrounding the generator.

There are three general methods of preventing trouble from frazil and anchor ice at the racks. These are: (1) heating the racks; (2) diverting the ice with compressed-air jets; and (3) the removal of the rack during ice runs.

Heating of Racks.—For the heating of the racks, the entire structure above water surface must be housed in, a baffle wall ahead of the racks, extending slightly below water surface, as in Figs. 190 and 213, being used to exclude the outside air. Warm air is then introduced inside the housing, and this warms the upper end of the rack bars, which should extend some distance above water surface into the housing. The rack bars should be continuous from top to bottom, and by conduction the heat is transmitted to the lower part of the racks. This method of heating is not entirely effective when the conditions are severe.

At the La Turque Development above Shawenegan Falls, on the St. Maurice River, Quebec, the racks are housed in and coils of steam pipes are placed in contact with their projecting tops. This device has practically solved the problem at that plant.

In some cases the rack bars have been heated by electricity. This method has been used considerably in Norway and Sweden where very extreme cold weather is experienced. It is said to be more economical and more convenient than steam.[16]

For heating the bars, either single bars or groups of bars are connected up in series and the current passed through them. It has been found that for bars having a cross-sectional area of 0.6 to 1.0 sq. in., sufficient heating will be obtained with 250 to 300 amperes per bar. In the walls of the intakes and of the ice chutes, electrical heating units are sometimes installed to heat up the concrete surfaces so that the ice will not stick to them.

Use of Compressed Air.—The installation for the use of compressed air is very similar to that described in Sec. 175 for removing trash from racks. For preventing ice troubles, however, the air is discharged from a pipe on the bottom of the forebay, located at such a distance above the racks that the air will rise to the surface, bringing the anchor and frazil ice with it, to pass over the top of the racks into a sluice of the type shown in Figs. 238 and 239 and

[16] Eng. News-Record, Vol. 93, p. 265.

described in Sec. 172. The proper distance from the foot of the racks to the location of the air pipe depends upon the velocity of the water, as it is essential that the air reach the surface before coming in contact with the racks, and also that it reach the surface near enough to the racks for the ice to be drawn into the sluice.

An installation of this type has recently been completed by the writer at the Deferiet Plant of the Power Corporation of New York, but has not yet been tested out. Others have reported successful use of this device.

Unless heating is resorted to, the racks should not project above water surface if anchor and frazil ice is expected to occur. An exposure of the upper ends of the racks to the cold air presents opportunity for radiation to cool the rack bars below the temperature of the water.

Removal of the Racks.—At the Holtwood Plant on the Susquehanna, the Cedars Rapids Plant on the St. Lawrence and at a number of other developments where there is no provision for preventing frazil and anchor ice from adhering to the racks, the upper sections of the racks are removed when ice starts to run.

At the Cedars Rapids Plant, the upper half of the racks in each bay is made in one section, to be readily removed as shown in Fig. 243.

Usually, little or no débris is present in the water when anchor and frazil ice is running, and the removal of the racks has caused no inconvenience or damage from that source.

CHAPTER XVII

CONDUITS

By William P. Creager and Joel D. Justin

177. Types of Conduits.—A general discussion of the use of both open and closed conduits is given in Chapter X. After the water has passed the intake at the dam, it enters the conduit and is conducted from there to the turbines at the power house. The general types of conduits used in hydroelectric practice are as follows:

- Open conduits:
  - Canals
  - Flumes
- Closed conduits:
  - Pipes
    - Steel pipe
    - Wood-stave pipe
    - Concrete pipe
- Tunnels

178. Location of Conduits.—*Field Method.*—The location of the conduit is sometimes made by methods similar to those frequently used in railroad location work. A field party for this purpose generally consists of a locating engineer, a transitman, a levelman, a rodman, two chainmen, two stakemen, and a field draftsman, together with as many axmen as the character of the country may require. As in railroad location work, the locating engineer divides his time between picking the country ahead and directing the party. The level party works ahead of the transit party, following the desired gradient or contour and carrying the stationing by pacing distances or by the use of a metallic tape. The transit party follows the level party, establishing the alinement and stationing the line. The level party doubles back and runs the profile over the established line. Usually, a good levelman can do this; but in some cases a second level party is used to run the profile over the stakes set by the transit party. If more than one line appeals to the locating engineer as being feasible, several alternate lines may be located.

Paper Method.—In conduit location for power projects, the exclusive use of the method just described is open to serious objection because all of the factors affecting the choice of conduit location cannot be known to the locating engineer in advance of the general office studies of the project. Thus the locating engineer might choose and survey a canal line, which in itself might

be a very good line, only to learn later that other considerations had dictated the location of the power house on the other side of the river or its removal to a location where a tunnel, instead of a canal, would be the logical type of conduit. For this reason it is much better to obtain topography covering the entire area in which one may be interested. With a topographic map giving the requisite amount of detail and the necessary field information as to the nature of the materials to be encountered, it is possible to make a paper location of the proposed conduit which is consistent with the other controlling factors of the project. Several alternative conduit lines can thus be laid down on the topographic map, their profiles plotted, costs estimated, and the most economical line determined. Then the field party should lay out and stake the line so determined and obtain a profile and cross-sections to substantiate the paper location. When the final location is made in the field, any changes that may then appear to be advisable can be made.

179. Limitations and Relative Advantages.—In the choice of a conduit, topographical and economic considerations will generally govern. A thorough investigation should be made of the feasibility of using various types of conduit for the project under consideration, and comparative estimates of cost should be prepared for the various types of conduit and alternative alinement.

Thus, for a certain project, it might at first appear that it would be a comparatively simple and cheap proposition to conduct the water by means of an unlined canal around the brow of the hill to a point just above the power house where the intake for the penstocks would be located. Further investigation and borings along the line of the work might show, however, that the material, through which the canal would pass, was a coarse gravel which would cause an excessive seepage, and that consequently it would be necessary to line the canal with concrete slabs. With this additional cost it is found, we shall assume, that the total cost of the canal line is greater than that of an alternative tunnel through ledge rock under the hill.

When the country is so steep and rugged that a conduit could not follow the hydraulic gradient, canals are, of course, ruled out of consideration, and the choice lies between steel pipe lines, wood-stave pipe lines, reinforced concrete pipes, and tunnels.

The conduits must be considered in connection with the other features of the project, as the various parts of the system are interdependent. Thus, it might pay to change the location of a dam site in order to shorten a conduit. On some projects the required length of conduit is found to be so expensive that it pays to break up the project into several smaller ones having a very much shorter total length of conduit. Generally speaking, a steep slope of the river and a high load factor favor the use of tunnels and pipe lines. A flat country and a low load factor favor the elimination of conduits, or their reduction to a minimum, and the location of the power house as an integral part of the dam. These features are treated in more detail in Sec. 82.

In cold climates, both canals and flumes are objectionable if of great length, on account of the problems arising from the formation of ice. Frazil ice, forming in the canal before it is covered with an ice blanket, often blocks the racks or the turbines. This trouble ceases when the canal is frozen over.

Flumes, on the other hand, are frequently designed for such high velocity that the surface never freezes, and more ice trouble may be expected. Pipes seldom give trouble from freezing; although, in many cases of exposed steel pipe, a coating forms on the steel inside the pipe and reduces the area somewhat. This ice coating, when it becomes loosened in warm weather, usually passes through the turbines without injurious results, but sometimes blocks the turbine gates completely. Concrete and wood are better insulators than steel, and freezing in concrete and wood-stave pipe seldom occurs.

In some localities, where the haul is long, it has been found that concrete pipe can be constructed at less expense than other types of pipe, particularly if stone and sand for concrete are near by. Concrete pipes, however, have not come into general use. They are limited to heads of about 250 ft.

Wood-stave pipe is usually less expensive than steel pipes for low heads, and is frequently adopted for heads up to about 200 ft. and to about 16 ft. in diameter. An arrangement frequently adopted for long pipe lines is to start at the forebay with a wood-stave pipe, carry this to a point where the head is such that a steel pipe is more economical, and use steel-pipe construction from this point to the power house. Wood-stave pipe has seldom been used for penstocks on account of the difficulty in anchoring and the uncertainty regarding water hammer.[1] The friction loss in a wood-stave pipe is probably less than that in any other form of conduit.

Steel pipe can be used for any head that can be developed, and to practically any diameter.

Tunnels are practicable in any size larger than about 4 by 6 ft. and are the most permanent form of construction. Their cost, however, is so great that they are never adopted unless conditions are such as to prevent economical installation of other types of conduits. They may be used under any head if they are deep enough in the ground, so that the weight of the over-burden is sufficient to balance the internal pressure, and they are sometimes reinforced with steel where the depth is insufficient.

180. Definitions for Pipes.—As explained more fully in Sec. 82, the pipe from the dam to the surge tank is termed the "pipe line," and the pipe from the surge tank to the turbine is termed the "penstock." In this and succeeding chapters, the term "pipe" will be used to designate both the pipe line and the penstock unless otherwise noted.

181. Number of Pipes.—For short distances between the intake and the turbines, a separate pipe for each turbine is the best arrangement for practical operation, because it permits of entering any one pipe without interfering with the operation of other units. For very short distances, this arrangement is frequently less expensive than a single pipe with Y branches at the turbines. For very long pipe lines and penstocks, however, economy requires a single pipe, except for the unusual case where the amount of water to be carried is so great that two or more pipes are required. Usually the surge tank is located close to the power house and, if the pipe line is long, it is a

[1] The fundamentals used in the determination of water hammer are not well known for wood-stave pipe.

single pipe, and a separate penstock is installed for each turbine unit in the power house.

A single penstock is provided with Y branches, as indicated in Fig. 247A, if it supplies a development consisting of two or more units.

Sharp bends, as indicated in Fig. 247B, are objectionable on account of excess friction loss, and are particularly bad when they are adjacent to the turbine as, in such cases, the eddies induced by the bend have a detrimental effect on turbine efficiency. The necessity for expensive branches at the end of a single penstock is an additional argument against the use of more than two units in the power house, if the penstock is so long that a single pipe is required.

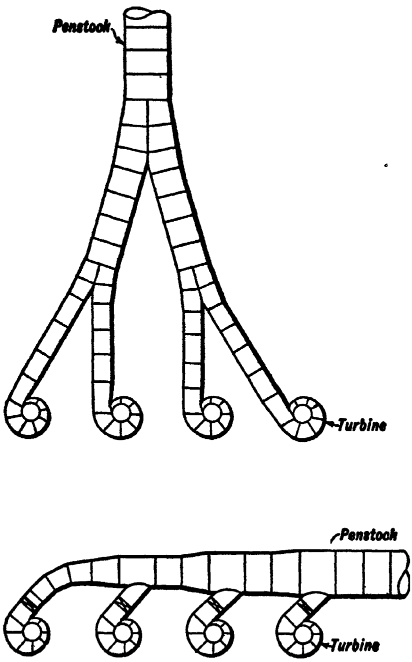


FIG. 247.

182. Air Inlets in Pipe Lines.—Under certain conditions of operation or partial failure of a pipe line, water may be removed from the pipe line faster than it can be supplied from the forebay. In such cases air must be admitted to the interior of the pipe to prevent the formation of a vacuum, with its consequent danger of collapse of the pipe. Such conditions are as follows:

(*a*) Should it be desired to empty the pipe line for any reason, the head-gates are closed and the contents of the pipe discharged through the turbine. Sketch *A* of Fig. 248 shows a typical pipe line and *EF* the hydraulic gradient for complete turbine discharge. When the headgates are closed and the water taken out through the turbine, the hydraulic gradient lowers as indicated in the figure, corresponding to the receding water in the pipe. It will be remembered that the discharge becomes less and the gradient flatter as the head on the turbine is reduced.

An air inlet, of capacity equal to the discharge of the turbine, is obviously required at the intake. For this typical pipe line, there is also required, at *D*, an air inlet of a capacity equal to the turbine discharge less the flow for a full pipe above *D* under a slope equal to *GD*. Obviously, if the slope *GD* corresponded to the slope of the hydraulic gradient passing through *D*, the flow above *D* would be equal to turbine discharge and no air inlet would be required. Hence it can be stated that, for this condition of operation, an air inlet is required at all points of change in slope, if the slope of the pipe above the point

is flatter than the hydraulic gradient. Incidentally, an air inlet would be required, at point B, equal to turbine discharge for head corresponding to water in the pipe at elevation B.

(b) The second condition requiring air inlets is that of a rupture at any point in the pipe line. Rupture of a pipe line is a rare occurrence; and it is usually considered that, except for very long and important pipes, this is not a necessary designing requirement. At any rate, the chances of a break or leak in the pipe line, in excess of turbine discharge, at times when the headgates are closed, is too remote for consideration.

Sketch B of Fig. 248 shows the conditions for a large break at the lower end of the pipe and with headgates open. The line $EDBA$, connecting the sharp breaks in the profile, is the hydraulic gradient under such conditions. This

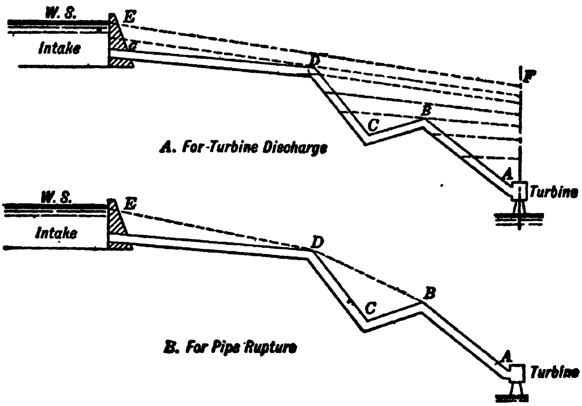


FIG. 248.

hydraulic gradient would fall below the pipe, causing a vacuum, if air inlets were not provided at points D and B. No air inlet is required at the intake. The inlet at D should be designed for the difference in pipe flow above and below D corresponding to the two gradient slopes ED and DB, and that for point B for the difference in flow corresponding to the two gradients DB and BA. If a break should occur at C the air inlet at B should correspond to the flow for the slope BC, and that at D to the difference in flow corresponding to the two gradients DE and DC above and below D.

Stand-pipes for air inlets are much to be preferred to air-inlet valves. Unfortunately, stand-pipes for inlets at points where the pipe is at a considerable distance below water surface are too expensive on account of the great height necessary, and air valves must be used. Stand-pipes are always used at the intake and are frequently simply a vertical hole in the intake concrete just below the headgates, as shown in Fig. 237. An air valve is essentially a vertical check valve of non-corrosive material.

Figure 249 shows details of one of the large air-inlet valves used at the

Davis Bridge Development of the New England Power Company.[2] This valve is described by Mr. Dow as follows:

With one exception vents where required on the New England Power System are of the open type. Below the Davis Bridge penstock valves, however, such vents would have had to be nearly 200 ft. high to avoid overflow under load changes, and while these could have been run up and supported on the surge tank, the question of accelerating the water column and of preventing freezing resulted in the choice of air valves. For this service valves having a 24-in.-diameter opening were developed and installed immediately

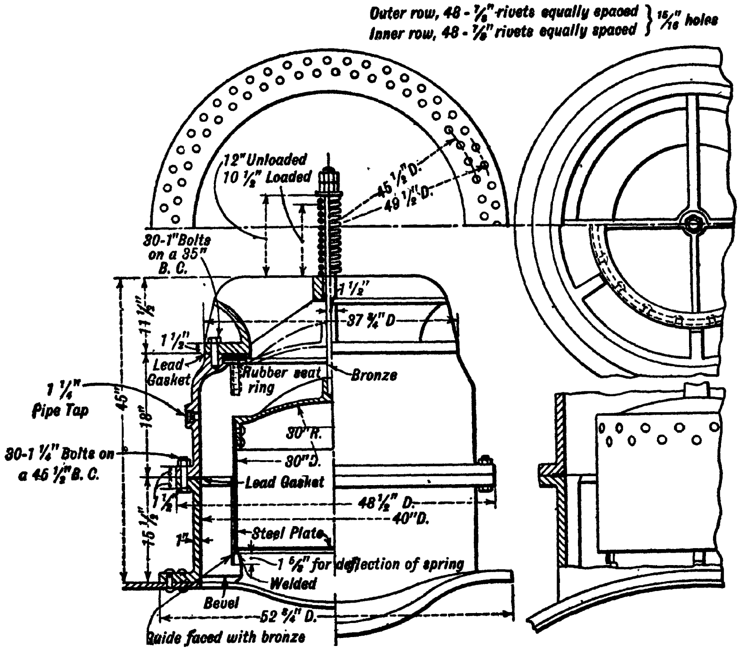


FIG. 249.—Davis Bridge Air-vent Vavle.

below the penstock valves. They are shown in Fig. 249, and, as will be seen, are closed by a float which rises as the penstocks fill. The bottom of the float is perforated to prevent collapse under the high pressure and also to reduce shock on closure by permitting the water to rise in the float and compress the entrapped air. The float is of bronze and rides loosely in bronze-faced guides. When closed a flat machined seat on the float bears against a ring packing in the housing. This ring is of hard grade and is made of a composition of rubber and duck. The valve is tight and shows no particular tendency to stick, though it opens with considerable suddenness when the pressure in the penstock drops. The opening is cushioned by the spring above the yoke at the top. All three valves are housed in a single compartment

[2] From "Mechanical Features Affecting the Reliable and Economical Operation of Hydro-Electric Plants," by E. A. Dow. Spring Meeting of The A. S. M. E., 1925.

provided with doors hinged to swing in either direction to admit or emit air when the valves operate. Freezing is easily prevented by the use of a few electric space heaters in the compartment.

Figure 250 shows a smaller air-inlet valve. A cluster of small valves, connected to the pipe by a manifold, have been used in place of a single valve of large size. Opinions differ regarding the relative advantages of a single large air-inlet valve, as compared with several small valves, for a single pipe. It is dangerous to place a gate valve between a single air-inlet valve and the pipe, as in Fig. 250, for the purpose of allowing inspection of the air valve during operation of the plant. The gate valve may be closed when the operation of the air valve is required. On the other hand, each valve of a cluster of smaller valves, of the same aggregate size, may be equipped with a gate valve and, if one of the air valves is a spare, any one may be closed off when desired. Moreover, a single air valve is much more likely to stick, through infrequent inspection, than are all the air valves in a cluster. However, if the air valve is properly designed and the pipe is emptied at frequent intervals so that the valve can be inspected regularly, a single air valve may be used.

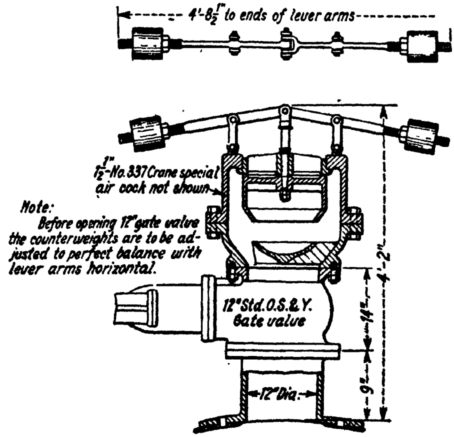


FIG. 250.—Assembly of 12-in.-Hawley Air Valve, Barrett Machine Co., Pittsburgh, Pa.

Air inlets of all types must be prevented from freezing. This feature is of the utmost importance. Air inlets of the type shown in Fig. 237 should be provided with a horizontal portion at the top to keep the water surface as far away as possible from the outside air, and should be provided with a housing similar to that described for the Davis Bridge valve. Inlets of this type, if well protected by a considerable thickness of concrete, do not need artificial heat to prevent freezing, as there is always a certain warmth in the concrete.

To determine the required size of air inlet,

Let Q = flow of air through air inlet, in cubic feet per second;
c = coefficient of discharge through the air inlet;
F = area of air inlet, in square feet;
P = safe difference in pressure between inside and outside of pipe, in pounds per square inch;
t = thickness of steel pipe in inches;
d = diameter of steel pipe in inches;
s = a factor of safety against collapse of pipe.

The following equation for flow of air, applicable to ordinary cases as explained later, is derived from an article by M. L. Enger and F. B. Seely:[3]

$$Q = 348cF\sqrt{P}.$$

Carman and Carr's equation [4] for the strength of steel pipe is:

$$P = \frac{50{,}200{,}000}{s}\left(\frac{t}{d}\right)^3.$$

Combining these two equations, we have for the safe area of the air inlet,

$$F = \frac{Q\sqrt{s}}{2{,}460{,}000c}\left(\frac{d}{t}\right)^{3/2}. \qquad (131)$$

For pipes buried in earth, a value of $s = 5$ should be used; and for pipes on saddles, $s = 10$ is a safe value.

Values of $c = 0.5$ for the ordinary type of air-inlet valves, and $c = 0.7$ for short air-inlet pipes, are conservative.

Owing to adiabatic expansion, the above formula becomes quite inaccurate (but on the safe side) when the difference in pressure between the inside and outside of the pipe line, P, exceeds 5 lb.[5] However, this is not particularly important as the permissible difference in pressure is much less than this for any but very small pipe lines. For instance, the collapsing-pressure equation shows that a 54-in. diameter pipe line with a thickness of steel of $\frac{1}{4}$ in. would collapse at a difference in pressure of about 5 lb. per square inch.

As an example of the application of Eq. (131),

$$\begin{aligned} \text{Let } Q &= 500 \text{ sec.-ft.;} \\ c &= 0.7; \\ t &= 0.5 \text{ in.;} \\ d &= 100 \text{ in.;} \\ s &= 10. \end{aligned}$$

Then from Eq. (131),

$$F = \frac{500\sqrt{10}}{2{,}460{,}000 \times 0.7}\left(\frac{100}{0.5}\right)^{3/2} = 2.6 \text{ sq. ft.}$$

183. Economics of Conduits.—High velocity, with resultant small area and small size of conduit, makes for cheapness in first cost, but results in high friction loss and decreased head and power output. Since minimum cost can be obtained only at a sacrifice of output and since maximum output can be obtained only at an increase in cost, there is always one size of conduit in every case which, theoretically, will result in the greatest economy of design.

[3] "Vents on Steel Pipe Lines," Eng. Record, Vol. 69, p. 594, 1914.

[4] Bulletin No. 5. Univeristy of Illinois Experiment Station. See also other experiments on collapsing pressure; E. E. Steward in Trans. Am. Soc. C. E., Vol. 27, p. 730, 1906.

[5] For $p = 5$ pounds, F is about 5 per cent too large.
For $p = 10$ pounds, F is about 50 per cent too large.

Hence, the theoretical best velocity in conduits is fixed by the principles of maximum economy described in Sec. 87. However, there are two practical considerations which influence the choice of velocity:

(a) In canals in earth, the velocity must not be high enough to cause scour or low enough to allow plant growth or deposits of silt.

(b) In penstocks, the velocity is influenced to a large extent by considerations of turbine regulation. A high velocity, while it may have been proved more economical, may require such a slow-moving governor, to prevent excessive water hammer, as to result in unsatisfactory speed regulation.

The usual velocities in conduits are described in Sec. 85. When the general principles of economic design, as given in Sec. 87, are applied to conduits, the following features must be taken into consideration.

Conduits in General.—The "annual cost" must include the annual cost of all appurtenances that change with the size of the conduit.

High-line Conduits.[6]—The "annual cost" of high-line conduits must include the cost of grading, sills, cradles, trestles, and other important appurtenances and must also include the annual cost of the terminal regulator. If the terminal regulator is a surge-tank, it will have some influence on the size of the conduit, because the size of the surge tank changes with the velocity in the conduit. If the terminal regulator is a pond created by a dam, the annual cost of the dam will vary, because the surges will be higher if the velocity in the conduit is higher. The internal pressure in closed conduits and the depth of water in open conduits are functions of the velocity in the conduits and of the size of the terminal regulator, and should be considered in all problems of economic design.

The problem of economics is somewhat complicated for the case of a closed conduit with a surge tank. For this case there are three variables: the charges against the surge tank, the charges against the conduit, and the value of power lost. For a given size of surge tank, the most economic size of conduit can easily be determined. Also, for a given size of conduit, the most economic size of surge tank can readily be determined. It will be found extremely difficult, however, to obtain by direct analysis the economic size of combined conduit and surge tank; in fact, this can be determined only by successive trials.

The problem is not so difficult for open conduits with ponds for regulators, as the cost of the dam creating the pond usually varies inappreciably with the size of the conduit.

184. Stresses in Closed Conduits.—The maximum internal pressure for which closed conduits must be designed consists of the maximum static water pressure, plus the maximum water-hammer pressure due to sudden reduction in the velocity, and the maximum excess pressure due to fluctuations in the water surface in the surge tank. The theory of water hammer is treated in Chapter XXIV, and the excess pressure in the conduit due to surges in surge tanks in Sec. 288.

[6] See definition of "high-line conduits" in Sec. 82.

Let T = the tension in pounds per linear inch of conduit on each side of the conduit;

H = the loading, in feet of water;

d = the diameter of the conduit, in feet;

p = the permissible stress in steel, in pounds per square inch;

A = the required gross area of steel per linear inch of conduit, on each side of the conduit;

E = the joint efficiency, expressed as a decimal.

Then

$$T = 2.6 Hd, \qquad (132)$$

and

$$A = \frac{2.6Hd}{pE}. \qquad (133)$$

For steel pipe, A is the required thickness of the plates. For wood-stave pipe, A is the area of the bands per linear inch of pipe, and, if the ends of the bands are upset so that E = 1.0, A is also the net area. For concrete pipe and banded tunnel linings, A is the area of steel reinforcement per linear inch of conduit.

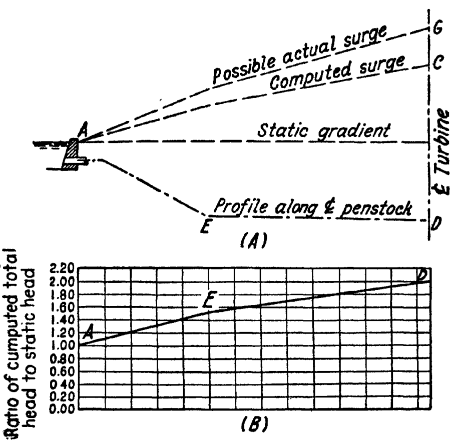


FIG. 251.

The stress in the steel members of any structure should never exceed the elastic limit. In the design of structures, the adopted unit stress for estimated loadings is chosen somewhat less than the elastic limit, to provide a margin against defects in fabrication and errors in the determination of the estimated loadings. Thus, in bridges and similar structures, the adopted unit stress is frequently about one-half the elastic limit. On the other hand, for cases where the loading is absolutely known, as in pipes of irrigation structures having no surges in flow and hence no pressures in excess of static, an adopted stress exceeding two-thirds the elastic limit is common.

The usual practice of adopting a low unit stress to compensate for inaccuracies in the loading assumption is not applicable to conduits for hydro-electro developments, since the *percentage* of surge, or load above static, is not constant at different points along the conduit. In Fig. 251A, the line AC is the computed surge gradient (water hammer or surge in pipe leading to a surge tank). The ratio of the computed total head to static head is indi-

cated in Fig. 251*B* where it is seen that the computed total head is 200 per cent of static at *D* and only 150 per cent at *E*. Consequently, a constant factor of safety to allow for errors in computed load would be erroneous, since an error of 50 per cent in the computed surge would result in an increase in stress of 25 per cent at *D* and only 16.7 per cent at *E*.

Therefore, it is recommended that the computed surge pressure be increased as in *AG*, Fig. 251*A*, by a constant percentage of the actual surge, to allow a margin for inaccuracies in its computation, and that the working unit stress in the conduit, based on gradient *AG*, be lower than the elastic limit only by an amount sufficient to allow for defects in fabrication.[7] In the event that this method is adopted, the stress at every point in the conduit will be equal if the gradient *AG* is reached, which is the objective of economical design, although the stress under static gradient or computed surge gradient may vary considerably at different points along the pipe.

The constant percentage to be added to the computed surge, in order to obtain gradient *AG*, varies with the existing conditions. The pressures due to surges in a surge tank are practically correct as the water cannot rise higher than the top of the tank. Pressures in pipe lines, due to water-hammer pressure, required to accelerate the water in the riser pipe of a surge tank, and water-hammer pressures in penstocks are not capable of exact determination. Recommended percentages are given in Chapter XXIV.

The stress to be adopted depends upon the probable defects in fabrication, the importance of the structure, and the amount of damage in case of failure. The following working stresses, based on gradient *AG*, are recommended.

Unimportant developments, where damage due to a break would be local only and would result in but a short shutdown for repairs	75 per cent of elastic limit
More important developments under the foregoing conditions	66 per cent of elastic limit
More important developments where the break would cause damage also to other structures, as for penstocks above a power house	60 per cent of elastic limit

The above working stresses are subordinate in concrete conduits to the desirability of relatively low stresses to prevent undue cracking of the concrete.

185. Bends in Closed Conduits.—Experiments [8] indicate that bends in closed conduits having a radius of about five times the conduit diameter cause the least head loss. Such bends are too short, however, for wood-stave pipe.

186. Bridges and Trestles.—Flumes and pipes are generally carried across small streams and depressions on bridges or trestles. Fills are objectionable on account of settlement. For the larger ravines and valleys, pipes are run down one side and up the other, forming what are frequently termed "inverted siphons." Such dips in pipes require blow-off valves [9] at the bottom, to take away deposits of silt and gravel, and air inlets [10] at the summits. Both

[7] It is not customary to allow for material corrosion of the metal of the conduit; but if it is desired to do so, a constant thickness may be added to the plates of steel pipes and a constant added to the diameter of the bands of wood-stave pipe, corresponding to the extent of corrosion anticipated.

[8] See Sec. 70.

[9] See Sec. 191.

[10] See Sec. 182.

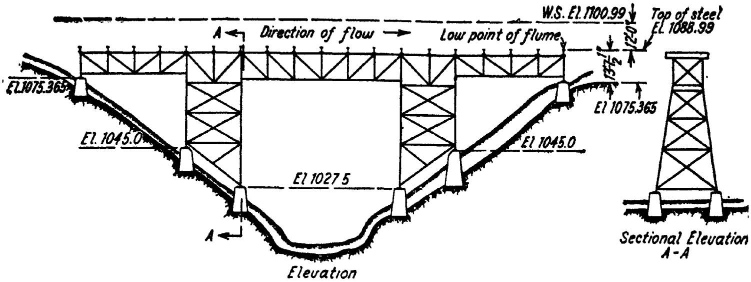


FIG. 252.—Viaduct No. 1, for Flume, Development No. 2, Tennessee Power Co., Ocoee Extension.

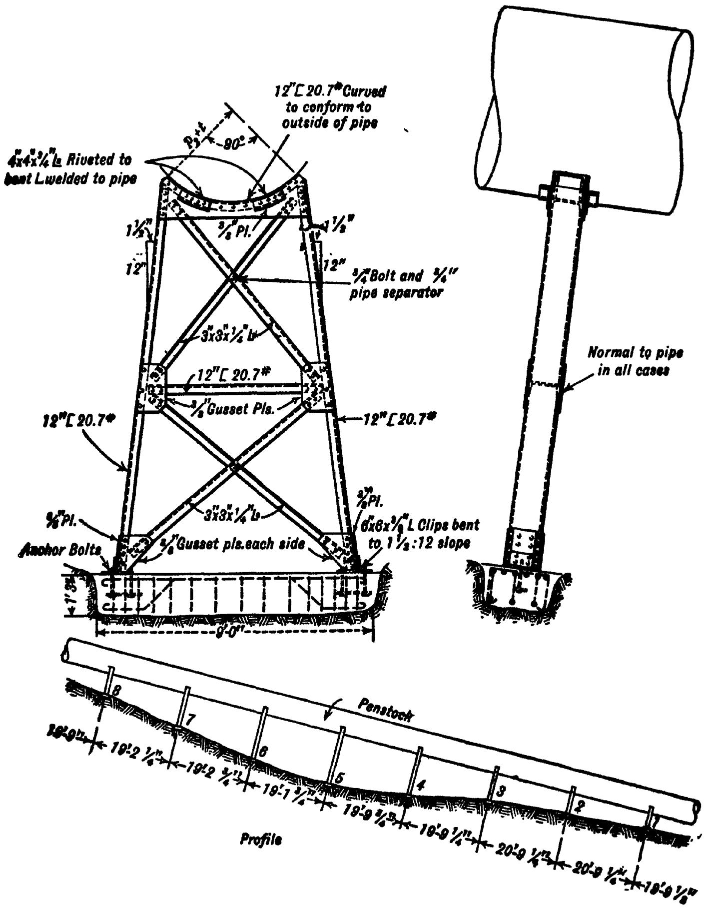


FIG. 253.—Detail of Steel-supporting Pier, Southern California Edison Co. From Report of Hydraulic Power Committee, National Electric Light Association.

of these are added features requiring attention, and the air inlets must be prevented from freezing in cold climates.

Details of wooden trestles are given in Sec. 209. Figs. 252 and 253 show typical steel trestles. Concrete trestles of various kinds have been used, but infrequently.

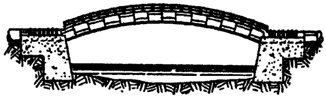


Fig. 254.—Arched Penstock Stream Crossing.

From "The Design of Steel Pipe for Hydro-Electric Plants," Richard Muller, p. 580, Engineering Record, Vol. 59.

Occasionally a steel pipe is arched between two masonry abutments to which it is anchored, so that the pipe itself forms the bridge. Such a case is indicated in Fig. 254.

187. Excavation for Benches for Flumes and Pipes.—Fig. 255 shows a typical side-hill cut for a bench for a wood-stave pipe. The slope of the excavation varies, of course, with the character of the materials. A width of 3 ft. 0 in. for ditch D is not too great for a deep excavation if sloughing is likely to occur. For rock cuts, the ditch is eliminated and the distance D can be reduced to about 1 ft. The width C, provided for sills depends on the type of saddles, but ordinarily will vary between $d + 1.0$ ft. for very small pipe to $d + 2.5$ ft. for large pipe. Distance C for steel pipe is, of course, less than that given above by the difference in thickness of shell, and for flumes it varies with the type of structure. The outside berm, B, should be at least 2 ft. for long, steep earth slopes on which the excavated material cannot be deposited. It can be reduced to zero if the excavated material can be deposited to the level of the bench.

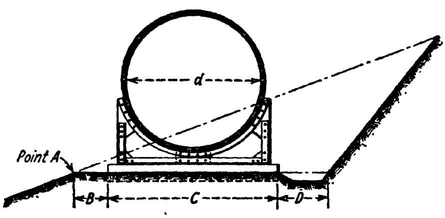


Fig. 255.

It is not common practice to rest the sills partly on bench and partly on side-hill fill, unless the fill has been deposited for a considerable period for full settlement.

Allowance should be made also for hand excavation of the trench for the concrete sill, if on earth cut. The sill is usually placed from 6 in. to 1 ft below grade. The estimated excavation for the bench should include full allowance for unevenness of the country through which the conduit is to go. This feature is often neglected.

188. Conduit Design.—The detailed design of conduits of the various types is given in succeeding chapters.

189. Manholes in Pipes.—A manhole should always be provided at each end of the pipe, and intermediate manholes located 1000 to 1500 ft. apart. The manhole at the intake is usually provided by installing a ladder in the air inlet, and that at the turbine is frequently furnished with the steel spiral casing. Intermediate manholes of various types are indicated in Figs. 256, 257 and 258.

190. Pipe-line and Penstock Valves.—Gates [11] are always used at the intake to permit of unwatering the entire conduit system for inspection and repairs, and valves occasionally are so used.

For low and moderate-head plants, the usual location of pipe-line and penstock valves is indicated in Fig. 259. If two or more turbines are supplied through a Y branch or a manifold at the lower end of a single penstock, as in Sketches *B* and *D*, a valve should be provided in each branch in order to permit of unwatering any one turbine during the operation of the others. For a long, single pipe line with short penstocks, as in Sketch *C*, a valve should be placed at the upper end of the penstock, so that it will be possible to unwater the turbine without having to empty the long pipe line and the surge tank.

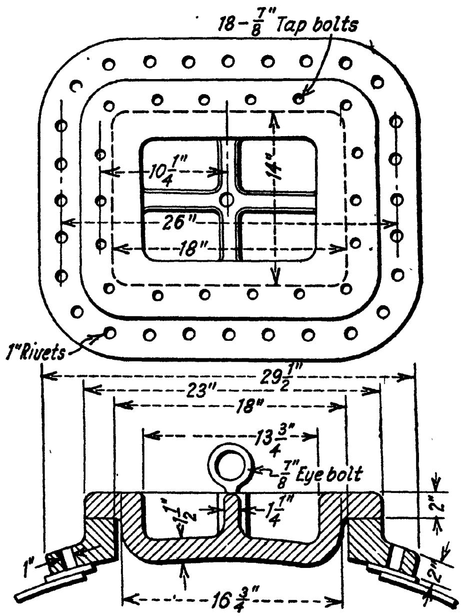


Fig. 256.—Manhole for Steel Pipe.

Leakage through closed turbine gates may be 5 per cent or more of full-gate turbine discharge after a few years of operation. If this leakage were allowed to occur, it would result in serious loss of water when the flow of the stream is no greater than that required for power. Consequently, since all types of modern penstock valves are practically water-tight, the turbines should be unwatered during periods of shut-down. If the penstock is long, as is usually the case for high-head plants, and provided with a valve only at its upper end, as in Sketches *A*, *C*, and *D*, the time required to empty it after the turbine is shut down, and to fill it when the turbine is to be put into service, may be so long as to justify the installation of a valve adjacent to the turbine, particularly if the conditions are such that the turbine must be placed on the line quickly. Such valves should be adapted to quick and easy operation. In this case, valves at the upper end of the penstocks, shown in Sketches *C* and *D*, would not be required unless, as in Sketch *D*, it should be desired to provide for unwatering one penstock while the other is in operation. This alternative, however, is seldom considered necessary, as it is unusual for the penstocks to need inspection more frequently than the pipe line.

[11] See Sec. 154.

Modern practice has limited closed-conduit valves to the following types:

(*a*) Gate valves;
(*b*) Butterfly valves;
(*c*) Needle valves, and
(*d*) Rotary valves.

All of these types of valves can be adjusted to negligible leakage, but the ease of such adjustment is greatest in the gate valve and least in the butterfly valve.

All valves, no matter what may be their type, when used for the control of water supplied to a hydro-electric plant, should be designed so that they may safely be closed under the full spouting velocity of the water through the valve, such as would exist in case of bursting of the casing of a turbine or of the penstock below the valve. Failure of the valve at this critical time may mean the flooding and entire ruin of the remainder of the station.

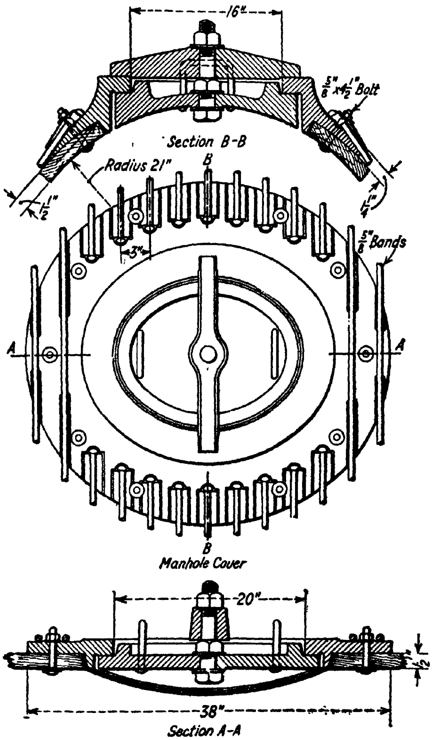


FIG. 257.—Cast Iron Manhole for 42-in Wood Stave Pipe, F. C. Kelsey, Consulting Engineer, N. Y. C.

Small by-pass valves are frequently provided to balance the pressure on the main valve by filling the conduit between the valve and the turbine, thus providing ease of operation. By-pass valves are

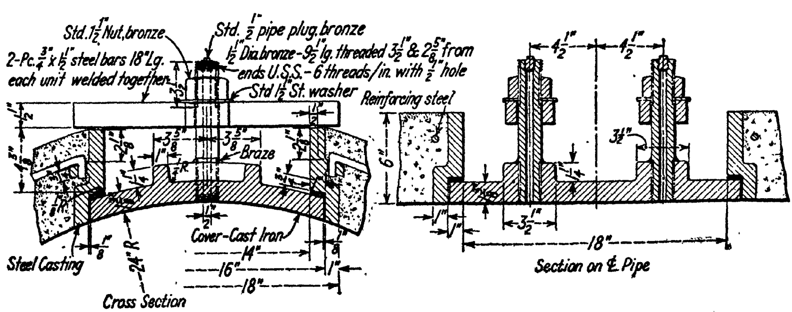


FIG. 258.—14" x 18" Manhole for 48-in. Reinforced Concrete Pipe. Lock Joint Pipe Co.

desirable for hand-operated installations and may be valuable if the main valve has a tendency to stick after being closed for a lengthy period.

The speed of operation of all conduit valves, if near the turbine, should be definitely limited to the speed of the turbine gates so that water-hammer pressure in the conduit will not exceed that caused by turbine-gate operation.

Gate Valves.—Gate valves have been used extensively in many plants, although their use is becoming less general at the present time because of the development of other types of valves which are more efficient and less expensive. Gate valves as large as 9 ft. in diameter for 100 ft. head and 28 in. in diameter for 2200 ft. head have been built. Fig. 260 shows a special 28-in., hydraulically operated gate valve built to operate under 2100-ft. head.

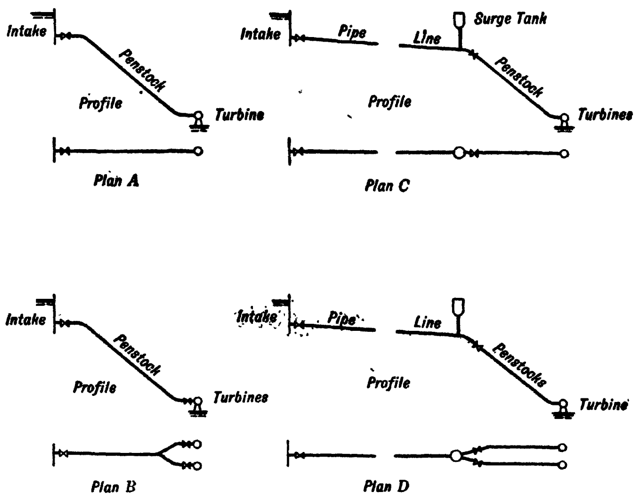


FIG. 259.—Arrangement of Valves for Low and Medium Head Plants

These valves were designed to be opened against full static pressure or to be closed against full-load velocity. In order to meet these conditions the plug of the valve must have a parallel seat and it must rest against this seat throughout its full travel. Valves for pipe-line conditions need have a seat on only one end, as pressure conditions are never reversed, unless for test purposes.

Gate valves have been practically superseded for diameters above 30 in. for medium-head conditions, although the small-size standard-type gate valves are still extensively used. The most important installations of gate valves at the present time are for high-head conditions, probably from 800 to 2500 ft. head and up to 30 in. in diameter. Valves for high-head conditions may be readily designed for penstock pressure operation by using a hydraulic cylinder direct-connected to the valve bonnet as in Fig. 260. The inlet water should be strained and the inside of the cylinder should be bronze-lined to prevent rusting. Some form of indicating device should be provided.

Fig. 260 shows a rod extending through the top of the cylinder. This serves the added purpose that when pressure is not available, as during erection, the valve plug or piston may be raised by attaching the crane to this rod.

For high-head conditions, both the face of the plug and the seat in the valve body should be renewable. A special design of gate valve for use with hydraulic installations has what is called a slot-filling ring, which is attached below the valve plug and is of the same diameter as the valve opening, so that when the plug is raised this ring fills the gap, making a perfectly smooth passage through the valve and thus reducing to a negligible quantity the friction through the valve. A by-pass is desirable on valves; but, for emergency conditions, they should be capable of being opened with full unbalanced pressure on the upstream face of the plug, and the area of the operating cylinder should be designed with ample margin to move the plug even though there be some sand on the face. High-head gate valves should be constructed of cast steel, and the parts subject to pressure are usually tested to about 50 per cent above the maximum head which will be obtained at the plant. Large-diameter gate valves have been practically supplanted by other types of valves, because of their cost, since the valve housing must be large enough to allow the plug to rise clear of the passage and these large chambers must be designed to resist full pressure. The standard type of gate valve is suitable for low heads and small diameters, but the valves should be selected carefully and have parallel seats, as a tapered-seat valve may be broken off when closing against full-load velocity, because under these conditions the plug is not supported and the full force of the water must be carried from the stem.

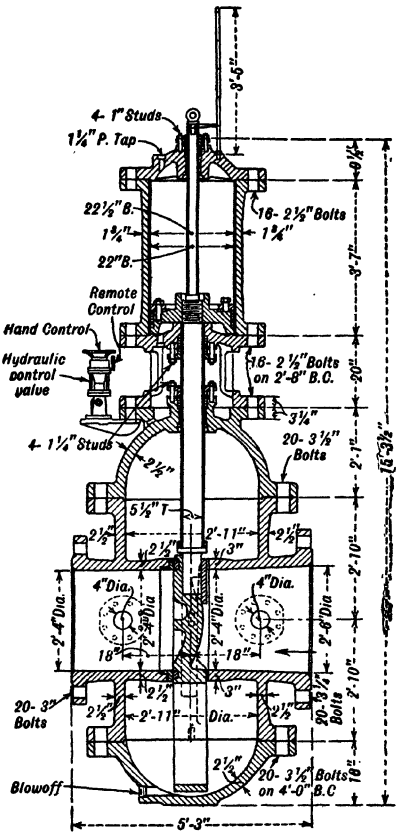


Fig. 260.—Section through 28-in. Gate Valve for 2100-ft. head.

Note parallel seat of renewable bronze, slot-filling follow ring below plug, separate adjustable stuffing boxes on both valve and cylinder, bronze protected rod, bronze lined cylinder, indicating and lifting rod extending through cover, electrically operated control valve on side.

Butterfly Valves.—The common type of butterfly valve, shown in Figs. 261 and 262 operates by torsion from a lever on an extension of the gate spindle. The wicket is tapered from both edges so that, when in the full open position where it lies parallel to the flow, it offers very little resistance or friction. Recently, large valves of this type have been provided with a form of piston ring set into the wicket, which, when the valve is in the closed position, is held out against the housing by water pressure or springs or both.

Usually the valve wicket is bored out and the pivot shaft extends entirely through the wicket and through the pivot bearings in the housing on each side of the wicket. This pivot shaft should be designed not only to carry the full pressure of the water on the face of the valve when in closed position, but also to take the maximum moment which will be exerted by the wicket when partly closed with the maximum velocity conditions existing in the pipe line. Owing to the uneven flow conditions set up around a butterfly valve when partly open or partly closed, the valve is not perfectly balanced as might be supposed; there is a strong closing tendency when the wicket is about 20 degrees

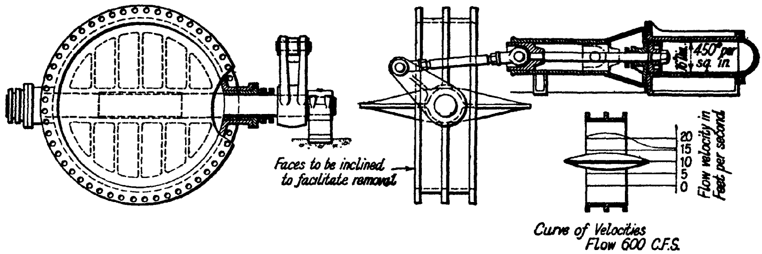


FIG. 261.—Typical Torsional Butterfly Valve. Aliis Chalmers Mfg. Co.

from the full open position. The entire operating mechanism, as well as the pivot shaft and wicket, should be designed to take this closing moment. The housing of the valve should be designed sufficiently rigid to retain very closely its circular form; otherwise the valve cannot be made tight. The housing should also be designed so that the internal pressure will not distort it so as to cause leakage. When in the closed position, half of the wicket is supported at the rim, since it is customary to design the valve so that it seats at from 10 to 15 degrees away from the diameter of the valve. The other half of the wicket, however, is not supported on the housing and will be deflected slightly on account of the pressure on this half. The design of the valve should take this into account, as there may be some leakage on this half if suitable sealing rings or some other means are not provided.

An interesting design of a butterfly valve is the disk-arm type shown in Fig. 263, where the operating lever is attached directly to the lower half of the wicket so that the closing force is exerted directly at the point where there is the tendency for the greatest deflection.

Butterfly valves may be operated either by a hydraulic cylinder or by a system of reduction gearing, driven by a hand wheel, a motor, or a small water turbine. The hand and motor-operating mechanism is used in the

majority of cases for medium and low-head installations; although under relatively higher head, where the pressure is sufficient and the cylinder area can be reduced to an economical size, the hydraulically operated mechanism has many advantages, as water pressure is always available in a power house while electric current is not, and in case of failure of the electric current it may take some five to ten minutes to close a large valve by hand. Hydraulic operating cylinders should be bronze-lined to prevent rusting and the cylinder

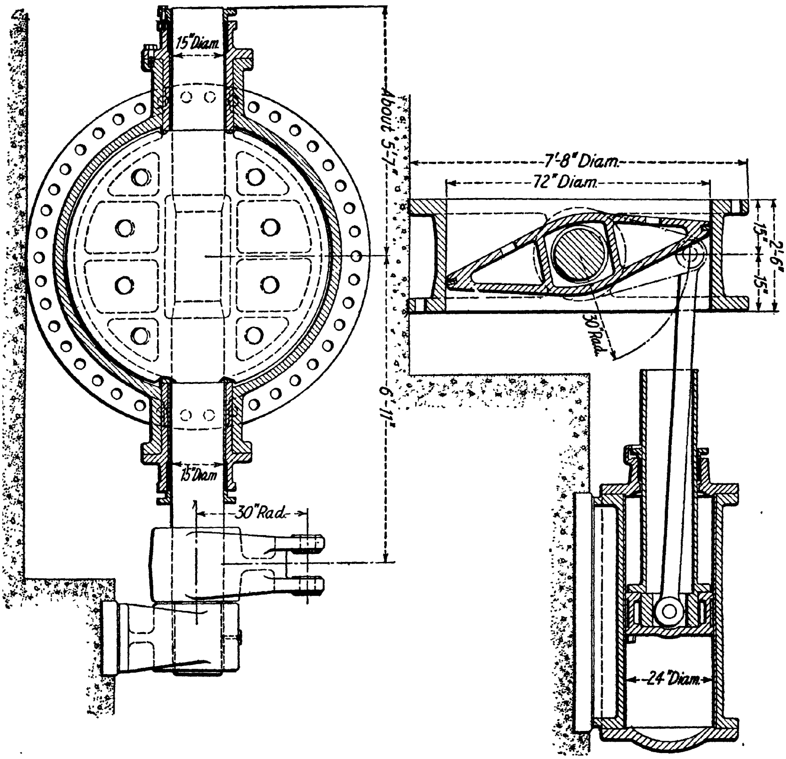


Fig. 262.—Typical 72-in. Torsional Butterfly Valve for Very High Head. Allis Chalmers Mfg. Co.

area should be sufficient to meet all conditions. There may be some tendency for the valve to stick in the closed position if it has been wedged tightly shut and held closed for a long time. A cylinder should be designed to meet this condition. The motor-operated mechanism is readily adapted to remote control; that is, the control switches for opening and closing the valve could be located adjacent to the valve, at the switchboard or at any distant point, but limit switches should be carefully adjusted so that the motor will be stopped when the valve reaches the full open or closed position. Care should be taken

on reaching the closed position so as not to allow the valve to jam. Frequently, indicating lights are provided so that the operator at the switchboard can know the position of the valve. Two lights should be sufficient, one for open and one for closed position, as there should be no need to use a valve in the partly open position.

Usually the pivot shaft is extended on both ends of the housing so that there is no end thrust on the shaft, and a readily adjustable stuffing box should

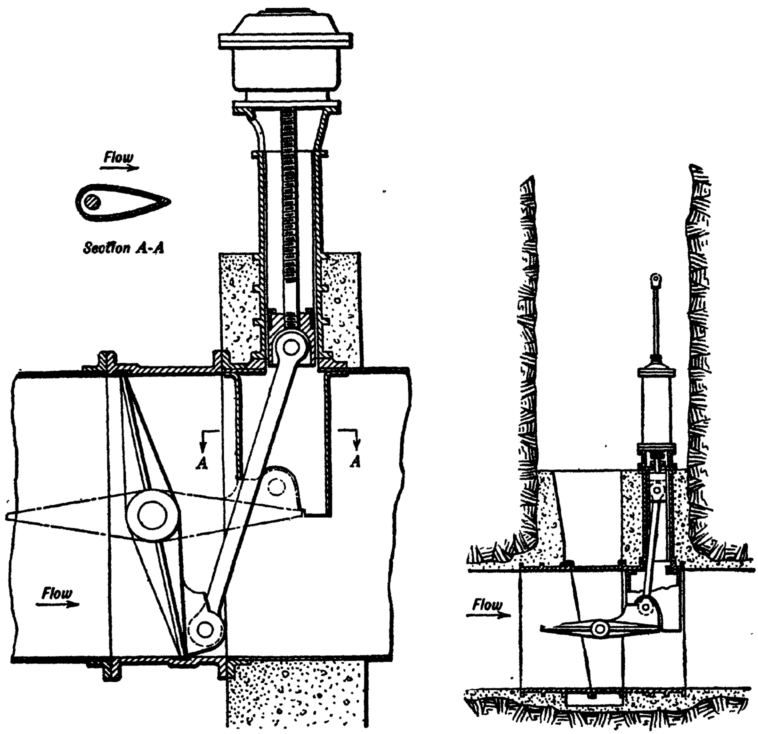


Fig. 263.—Typical Dow Pivot Valves. From Catalogue of S. Morgan Smith Co.

be provided at each end. The trunnions should be provided with renewable bushings, and some provision should be made for forcing grease into these bearings. While by-pass valves are usually provided on the larger sizes, they are not necessary for medium heads, as it is an easy matter to open these valves a slight amount. Plants having long penstocks usually have some form of valve at the lower end of the penstock where it connects to the turbine casing, and frequently another valve is located at the upper end of the pipe line. These upper valves sometimes take the place of the head-gates.

Control of Valves.—The following systems of control may be applied to all types of conduit valves except the needle valve. The least expensive is hand control through gears to a hand wheel. This type of control is commonly used on small valves, and in cases where it is not necessary to close the valves quickly.

Motor control is similar to the hand control, except that a motor is geared to the operating mechanism. This motor may be put into operation in either direction, from a control panel adjacent to the valve or a control panel on the switchboard; or it may be designed to be set into operation by any protective device which will make a contact in case of low pressure in the penstock, such as would be caused by a bursting of the pipe line or casing. With the motor-operated mechanism there is some danger due to the possibility of failure of the current supply, and this must be taken into consideration when designing the control system. An interesting development is the use of a small impulse turbine which is geared to the mechanism in the same manner as the motor, and operated by water from the penstock.

Hydraulic-cylinder control is used extensively on large gate valves or butterfly valves when operating under high heads; but for low-head conditions, the hydraulic-cylinder mechanism becomes very expensive, because of the large area of cylinder required to obtain the necessary closing forces with the low-water pressure. Hydraulically operated gate valves are in successful operation under heads as high as 2200 ft. Hydraulic operation, while it is slightly more expensive than the motor-operating mechanism for butterfly and gate valves, has certain advantages because of its simplicity and smoothness. The cylinder should be bronze-lined to prevent rusting, and a filter of some form should be placed in the supply pipe to prevent foreign matter from the penstock from clogging the control valve.

Valves with hydraulic-cylinder control may be provided with a pilot or control valve which is motor operated, so that the valve may be opened or closed by electrical control from the switchboard or a remote point. The controlling pilot valve should consist of a double-port balanced valve which connects one side of the cylinder to the pressure supply and the other side to the discharge.

All valves, while they may be equipped with by-passes, should be designed to open with the full pressure on the upstream side of the valve, which may be necessary in an emergency in starting up a unit or in case of failure of the by-pass. This can be readily accomplished with the butterfly valve or needle type, but gate valves must be specially designed to meet this condition.

For additional data on operation of valves, see Sections 161 and 169.

Needle Valves.—The following description of the Johnson type of needle valve was furnished by the Larner Engineering Company.

Johnson Hydraulic Penstock Valve Type "B"

This design of Johnson valve is used almost exclusively in penstocks of water-power plants, and is invariably placed close to the entrance of the wheel casing. Its form, wherein the inlet is larger in area than the outlet, makes it particularly suitable for connection to the wheel casing, as the outlet end can

be made the same diameter as the wheel casing inlet and the inlet end of the valve connected directly to the penstock.

The waterway through the valve is formed with stream-line passages which permit of relatively high velocities and low hydraulic losses. The operation of the valve is accomplished by the use of the pressure of the penstock, there being no occasion to resort to outside sources of power. The differential character of the plunger results in two operating chambers, a central chamber " A " and an annular chamber " B." The valve plunger is closed by admitting pressure from the waterway to chamber " A " and exhausting pressure from chamber " B " to the atmosphere. It is opened by a reversal of this operation. The actual interchange of pressures in the operating chambers is accomplished by various control devices which are suited to the particular requirements. Usually no electric power of any sort is used, except for remote indication of the position of the valve plunger. The principal characteristics of the control for this type of valve are as follows:

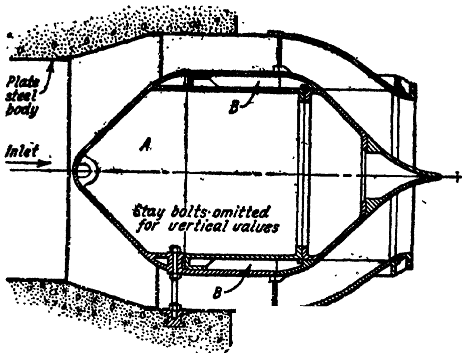


Fig. 264.—Type "B" Johnson Hydraulic Valve, (Patented)

(1) The valve can be operated just as readily in flowing water as when there is no flow.

(2) The rate of closure or opening can be set permanently when the valve is installed, or can be altered at the will of the operator. The movement of the valve plunger is under absolute control at all points of its stroke.

(3) The wheel casing beyond the valve can be primed by opening slightly the main valve plunger. This priming feature can be made automatic, in that a single movement of the control handle will first prime the wheel casing and later complete the opening stroke of the valve.

(4) The valve can be arranged to close automatically in case of a break of any nature beyond the valve which would cause a flow in excess of normal.

(5) The operating characteristics permit the use of the valve as a protective device. Generally the location of the valve is in the power house, but in some instances, where protection for the penstock is desired, the valve is located in the penstock above the power house.

(6) The operation of the valve can be performed at the valve or from a remote point, or both.

(7) The valve is absolutely tight when closed, as the plunger seats against a machined ring in the neck of the valve body.

(8) The circular form of the valve permits its being manufactured in practically any size or for any pressure within the present range of penstocks for hydro-electric power plants.

Type " E " Johnson Regulating Valve

This valve is designed primarily for free-discharge service where water is discharged directly to atmosphere, or where the valve is to be throttled to control the flow from a region of high pressure to a region of low pressure.

It is further used in smaller sizes for penstocks of hydro-electric plants,

particularly for extremely high heads. It differs from the Type " B " Johnson valve primarily in the nature of the control, the general form of the valve and fundamental means of operation being the same.

The valve is normally operated in dead water by pushing the plunger mechanically, while in flowing water the mechanical work is confined to moving the pilot valve which controls the pressure in the central operating chamber. The principal characteristics of this valve are as follows:

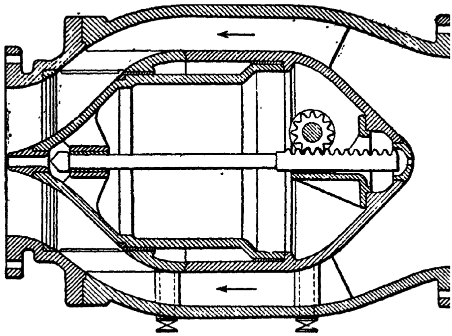


Fig. 265.—Type "E," Johnson Valve.

(1) The shape of the valve, tightness when closed, lack of hydraulic losses, arrangements for local and remote control and similar characteristics are the same as given for the Type " B " valve.

(2) This valve is so arranged that it will remain at any intermediate position, without tendency to move, for an indefinite time.

(3) Where it is used for free-discharge service the shape of the plunger and valve body are arranged to permit of continuous discharge through partial valve openings without undue wear or erosion of the surfaces.

(4) The plunger is in absolute hydraulic balance at all points in the stroke, this being the most effective way to hold the plunger in any desired positions and prevent vibration.

(5) The valve is equipped with auxiliary blow-offs from the two operating chambers, so that in dead water the operation can be performed similarly to the type " B " valve. This is seldom resorted to as the mechanical strength of the control is sufficient to push the plunger back and forth in dead water.

Rotary Valves.—Escher Wyss & Co. of Zurich, Switzerland, have recently put on the market a new type of rotary valve which is adapted for use in pipe lines and penstocks connected to hydraulic turbines.

By reference to the cross-sections of the rotary valve, illustrated in Fig. 266, it will be seen that the casing is spherical in form, which makes it equally suitable for all sizes and pressures. The casing has two easily lubricated trunnion bearings by which the rotary part of the valve is supported so as to rotate through 90 degrees.

Passing through the rotary part is a clear, smooth, cylindrical opening of the same diameter as the pipe to which the valve is connected. At 90 degrees from the opening, in the plane of rotation, the rotating part is bored to receive a disk, *A*, which slides in the rotating part and is beveled to match a seat surrounding the outlet of the casing. A small pipe connection or by-pass, with a valve, *B*, permits the pressure on the two sides of the disk to be equalized.

To close the valve, the rotating part is turned through 90 degrees to bring the disk opposite the outlet of the casing, in which position there is but little flow through the valve. The valve, *B*, in the small connecting pipe, or by-pass, is then closed, allowing the pressure which exists in the casing to accu-

mulate behind the disk and force it to its beveled seat, tightly shutting off all flow through the rotary valve. The water reaches the space behind the disk through the small clearance between the turned circumference of the disk and the bore in which it is fitted.

To open the valve, the by-pass is first opened, releasing the water from the chamber, *C*, faster than it can enter through the small clearances, thus equalizing the pressure on both sides of the disk, so that it does not bear upon its seat. The valve is then turned 90 degrees to its open position.

The rotation is accomplished by means of simple gearing, which is located outside of the casing and may be operated by hand or motor or by hydraulic cylinders. When it is hydraulically operated, an electric-motor-operated actuator is sometimes used to control the main valve and the small by-pass in proper sequence, thus permitting distant control by means of a small electric switch.

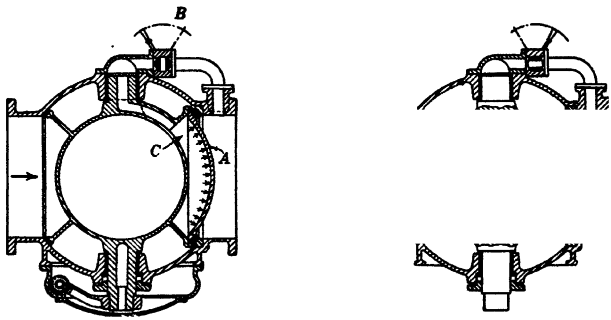


Fig. 266.—The Escher Wyss Rotary Valve.

The rotating part does not bear against the casing, except at the lubricated trunnion journals, thus allowing the valve to be rotated with but little effort. When the valve is closed the entire pressure is carried by the disk.

While the Escher Wyss & Co. rotary valve is best adapted for use in pipe lines up to any pressure likely to be encountered in water-power installation, and has thus been installed under heads of 100 to 3240 ft., it has also been used as a sluice valve in the base of dams under heads up to 150 ft. While the valve will open and close when discharging into the atmosphere, it is not to be recommended for such service, under any but moderate heads, by reason of the disturbance in the discharge at partial opening.

Summing up, the advantages claimed for the Escher Wyss & Co. rotary-type valve are:

(*a*) The parts subjected to water pressure are spherical or cylindrical in form, which gives safety against bursting, with the use of a minimum amount of material.

(*b*) It can be easily opened or closed under all operating conditions without slamming, by hand, by motor, or hydraulically, with oil or water pressure, and distant control if desired.

CHAPTER XVIII

CANALS

By William P. Creager and Joel D. Justin

192. General.—A general outline of the purpose and use of canals in hydro-electric developments and a discussion of those features of canals which are common to all types of conduits are given in Chapter XVII.

193. Shape of Section.—Several types of canals are shown in Fig. 267. There are, of course, many other types for special conditions. Canals in earth generally have a trapezoidal section, while canals in rock frequently have very nearly vertical sides, the slope of the sides being determined by the angle at which the rock breaks out most conveniently.

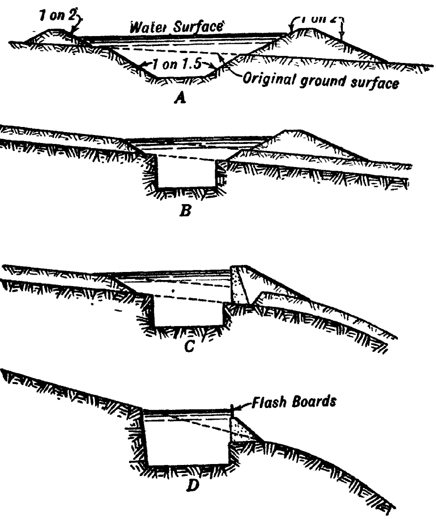


Fig. 267.—Several Types of Canals.

Canal walls, as in Fig. 267*C*, are used only where sufficient room for an embankment is not available. The canal wall shown in Fig. 267*D* is, in reality, a dam, as it has no back fill. The danger of ice thrust between the rock face on one side and the concrete on the other is a source of danger and has caused several failures. For this reason, flash-boards have been installed to bend with the expansion and contraction of the ice and relieve the concrete from most of the thrust.

With given area of cross-section and given slope, the form of cross-section that will give the greatest hydraulic radius will pass the greatest discharge and will therefore be (theoretically) the most economical form of cross-section. Thus, a semi-circle would be the most economical form of cross-section, theoretically, but would be impracticable to excavate. It may be shown that for

maximum hydraulic radius, or minimum wetted perimeter, for a trapezoidal section as in Fig. 268

$$d = \sqrt{\frac{A \sin \theta}{2 - \cos \theta}},$$

$$b = 2d \tan \frac{\theta}{2},$$

$$r = \frac{d}{2}.$$

In which,

A = area of water cross-section;
θ = angle which the side slopes make with the horizontal;
P = wetted perimeter;
r = hydraulic radius $= \frac{A}{P}$;
d = depth of water.

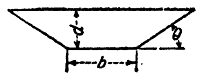


FIG. 268.

For vertical sides, as in a rock cut, θ is 90 degrees and the bottom width for greatest hydraulic radius is twice the depth of water.

The foregoing theoretical considerations must be modified in practice, as other questions of economy usually will not allow the use of that section which has the maximum hydraulic radius. Thus, in a deep rock cut, the theoretical ratio of bottom width to depth of water must be reduced to decrease the amount of total excavation. Fig. 269 shows two canals having equal friction loss per linear foot and equal discharge. Canal A is designed for the greatest hydraulic radius, while Canal B, having a slightly larger water area, results in considerably less excavation. It will frequently be found, also, that the bottom width may more economically be made wide enough for steam shovel and track work, even at the expense of additional excavation.

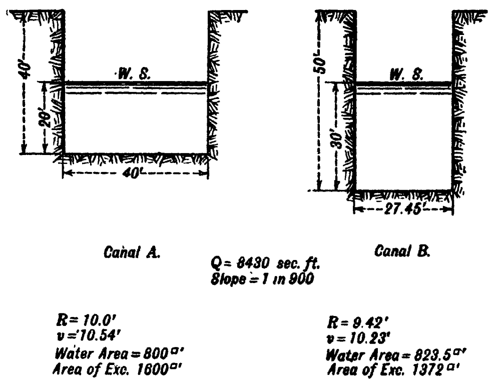


FIG. 269.

In the same way it will be found that deep-cut and side-hill canals in earth will have a most economical section which does not correspond to that having the greatest hydraulic radius.

The section of canals is frequently influenced by the necessity for providing against ice troubles, as indicated in Sec. 194.

194. Ice Trouble in Canals.—Shallow canals in cold climates sometimes give a great deal of trouble in the winter time because of ice. This is especially true if the canal is long as well as shallow and the water flows at a relatively high velocity. Under such conditions the velocity is high enough to prevent the canal from freezing over to any great extent, and the result is that "frazil" or needle ice often forms in considerable quantities, giving a great deal of trouble at the racks and sometimes at the unit itself, and frequently causing a shut-down of the plant. For these reasons, some other type of conduit is sometimes selected, in preference to a canal, for a project in a cold climate, even where a canal would apparently be the most economical.

However, ice troubles in canals can be obviated, or at least reduced to an insignificant minimum, by suitable design, construction, and operation. It is, therefore, not sound judgment to omit entirely the consideration of their use in a cold climate when a real saving could be effected thereby.

Narrow, deep canals are subject to less ice troubles than shallow, wide ones of the same capacity.

Other considerations, such as maximum economical loss of potential energy, and maximum velocity for safety against erosion for the material through which the canal passes, will generally limit the maximum velocity in the canal cross-section to 3 or 4 ft. per second. Under such conditions, the velocity at night, when the load on most hydro-electric plants is at a minimum, will be but a small part of this; and the ice sheet will usually form before there is very much trouble from frazil or anchor ice. Generally speaking, the minimum velocity in the canal should be less than 1.5 ft. per second in order to permit the formation of an ice sheet. With paper mills and chemical plants having a large night load, it is frequently desirable, at the beginning of a cold snap, to shut down or run light for a night or two in order to permit the ice sheet to form. Once an ice sheet has formed on a power canal, there is seldom any further trouble from frazil or anchor ice.

Assuming that the construction of the intake to the canal is such that ice cakes are not permitted to enter the canal, ice should rot out of a properly designed and constructed canal without causing very much trouble. When the intake is located in a river that carries large quantities of cake ice, like the St. Lawrence, Niagara, and Susquehanna, the water is sometimes drawn from the bottom of the river at relatively low velocity. The direction of the current to the intake is often approximately at right angles to the surface current. The surface current of higher velocity then carries the cakes of ice past the intake. This arrangement is sometimes facilitated by excavating grooves in the bottom of the river to aid in conducting the lower stratum of water to the intake.[1] Provision should also be made in the forebay, near the intake to the penstocks or power house, by means of a skidway, to take care of cakes of ice.

195. Economics of Design.—For successful operation, the size of the canal and its velocity for a given discharge may vary between wide limits; but in some cases, particularly in lined and rock canals, there is one size only, for

[1] See Ice Diversion, Hydraulic Models, and Hydraulic Similarity, by Benjamin F. Groat, M. Am. Soc. C. E. Trans. Am. Soc. C. E., p. 1138, Vol. LXXXII (1918).

each case, which will make for greatest economy of design. The reader is referred to Sec. 87 for the general theory of economic design and to Sec. 183 for its application to the special case of conduits. A typical investigation of the economics of design for the Chippewa Canal at Niagara Falls is given in *Eng. News-Record,* Vol. 85, p. 744.

196. Permissible Velocities in Canals.—*Velocities to Prevent Sedimentation.* —The velocity of water in a power canal should not fall low enough to permit the deposit of silt or débris. The minimum limiting velocity need not generally be given much consideration, because, as a rule, the intakes for power canals are located at the lower ends of ponds or reservoirs. That portion of the débris carried by the stream which is fine enough to be readily deposited will be dropped in the pond near the point where the rapidly moving water of the stream comes in contact with the quiescent water of the pond. Hence, when the intake is from a pond or reservoir, material in suspension that has failed to settle out in the quiet water of the pond will not settle out to any extent in the comparatively rapidly moving water of the canal.

However, even in such a canal, if the water is laden with silt or clay and the plant is shut down, the canal, for the time being, becomes a settling basin and some of the silt and clay will be deposited. One might think that when the plant was started up and the usual velocity re-established in the canal, the silt and clay thus deposited would be picked up again. But this does not often happen, for the reason that the transporting velocity and the eroding velocity for any given material are often quite different. For sand and gravel the eroding and transporting velocities are not very different, but for clays and silt the margin is wide. Thus, if coarse sand or gravel is deposited in a canal, as soon as the velocity of the water has increased to just a little more than it was when the material was deposited, the sand or gravel will be picked up again and transported with the water. On the other hand, if clay or silt is deposited, the velocity of the water must increase very much above the velocity at which the deposit took place, before the material will be picked up again.

A mean velocity of from 2 to 3 ft. per second will generally be sufficient to prevent the deposit of silt. The subject has been given a great deal of study in India, and R. G. Kennedy [2] gives the formula:

$$v = cd^{0.64},$$

which is based on extensive experiments. In this formula,

c = a coefficient which varies with the character of the silt between 0.82 and 1.07;
v = mean velocity in canal which will prevent the deposit of silt;
d = depth of water in feet.

Velocities to Prevent Plant Growth.—In some climates, the growth of aquatic plants and moss seriously affects the capacity of canals. It has been found

[2] "Remodeling of distributaries on old canals," Punjab Irrigation Branch Papers, No. 10, J. S. Beresford, R. G. Kennedy, and T. Higham.

that when the temperature of the water is below 65 degrees Fahrenheit, algae and moss growths are not serious, and also that the growths do not take place to any extent in turbid or deep water. A mean velocity of not less than 2.5 ft. per second will generally prevent a growth that would seriously decrease the carrying capacity of the canal.

Velocity to Prevent Erosion.—The maximum permissible velocity in a canal is the greatest velocity that will not erode the bottom and sides of the canal. In most cases this limiting maximum velocity will determine the cross-section of an unlined canal, because an unlined canal that meets the criterion of economic conduit design given in Sec. 87 would have a velocity high enough to erode the bottom and banks of the canal. Velocities frequently used in unlined canals are from 2.5 to 3.5 ft. per second. If the soil contains a considerable percentage of clay or gravel, velocities as high as 5 ft. per second may be used. When a canal is lined with concrete, no definite maximum limiting velocity can be set, provided that the water does not carry sand, gravel, or stones. Velocities of 40 ft. per second were successfully used in a spillway chute of the Strawberry Valley Irrigation Project in Utah.

In using very high velocities over a concrete lining, as is sometimes done in a spillway canal or channel, it should be remembered that there is a tendency for the rapidly moving water to pick up the blocks and move them out of position, because the pressure under the blocks sometimes exceeds that of the rapidly moving water just above. For this reason, when high velocities are used in such a channel, the concrete blocks are often made several feet in thickness. This is especially desirable when the foundation on which the concrete blocks are laid consists of sand, gravel, or soil, instead of rock.[3]

Table XXXIX gives Etcheverry's recommended maximum mean velocities safe against erosion. Bottom velocities are approximately 75 per cent of mean velocities. Etcheverry's limit of concrete probably applies to thin linings, as mass concrete can withstand much higher velocities if properly built.

TABLE XXXIX

MAXIMUM MEAN VELOCITIES SAFE AGAINST EROSION

	Velocity in Feet per Second
Very fine sandy soil or loose silt	0.50
Pure sand	1.00
Light sandy soil, 15 per cent clay	1.20
Light sandy loam, 40 per cent clay	1.80–2.00
Coarse sand	1.50–2.00
Loose gravelly soil	2.50
Ordinary loam	2.50
Ordinary firm soil or loam, 65 per cent clay	3.00
Stiff clay loam	4.00
Firm gravelly clay soil	5.00–7.00
Stiff clay	6.00
Conglomerates, soft slate	6.50
Stratified rocks	8.00
Small boulders	8.00–15.00
Hard rock	13.33
Concrete	15.00–20.00

[3] See "Irrigation Practice and Engineering," by B. A. Etcheverry, McGraw-Hill Book Company, Vol. 11, p. 55.

197. Side Slopes.—With an unlined canal, the side slopes should be determined by the slope at which the material will permanently stand under water. Usually, the slopes used in cut may safely be steeper than the slopes of the same material in fill. Thus, in Fig. 267*A*, it is assumed that the material with which we have to deal will stand in cut on a safe under-water slope of 1 on $1\frac{1}{2}$. Then the under-water slope for the same material, in embankment, should probably not be steeper than 1 on 2 for the same factor of safety against sloughing. When the material is sand and gravel or a very sandy soil, having but little cohesion, this slope differential does not apply, and the same slope may successfully be used for the water face of the embankment as in the cut. Except for canals in rock, where vertical or nearly vertical sides are used, the side slopes of unlined canals should seldom be steeper than 1 on $1\frac{1}{2}$, and never steeper than 1 on 1.

A discussion of side slopes required for lined canals is given in Sec. 200.

198. Design of Canal Embankments.—For very short power canals, it is usually economical and desirable to make the elevation of the top of the canal well above the elevation of high-water surface in the pond. This obviates the necessity of using control works at the entrance to the canal and provides for full head on the turbines during periods of high water. With longer canals this practice becomes uneconomical, and control works are located at the entrance of the canal, with gates in them, so that the water level in the canal can be controlled in case there is an increase in the level of the water in the pond from which the canal takes its water. In such cases, the level of the water in the canal is also controlled by the use of one or more spillways along the canal. In determining the proper free-board to use for a canal, the surge, due to a sudden shutting down of the turbine gates, should be computed and allowed for,[4] and due allowance should be made for possible errors in computing the depth of flow, as described in Sec. 74.

Canal embankments are merely small earth dams; and the same principles which apply to the free-board above highest surge, the width of top, the slopes, rip-rap, cut-offs, etc., in earth dams, are applicable to the design and construction of canal embankments.

199. Seepage Losses in Canals.—In canals in arid regions, seepage losses are a matter of great concern; but in most power canals, the losses due to seepage are rather insignificant. The seepage of water from canals is discussed in Sections 24 and 114.

A canal that is in cut, other things being equal, will have much less leakage than one that has a great deal of embankment. It is frequently possible to put the canal in cut below normal water surface, and provide embankments to retain unusual high water and surges.

200. Lined Canals.—When canals are constructed through pervious material, they are sometimes lined with slabs of concrete or other impervious material in order to prevent the leakage which would otherwise take place.

It should here be noted that the excavation of the canal cross-section for a lined canal will, in most cases, be much less than the excavation of an unlined canal for the same project, for the following reasons:

[4] See Sec. 78 for method of computing the height of surge in a canal.

(1) A lined section will give a much lower value of Kutter's n than an an unlined section.

(2) A lined section will permit the use of a velocity giving an economical loss [5] of head, instead of a velocity limited to the maximum for which the given material is safe against erosion.

Concrete Slab Lining.—Concrete slabs are frequently used for lining power canals. For this purpose the concrete is generally mixed in the proportions of 1 : 2 : 4, or even richer. One method is to use concrete blocks or slabs from 4 to 6 in. thick and from 5 to 12 ft. square. The purpose of dividing the lining up into blocks is to provide for the possibility of settlement or heaving of the material beneath. If this should occur, the blocks would move as units, and not produce cracks as a monolithic lining would do. The division into blocks also provides for expansion and contraction due to temperature changes. Sometimes these blocks are poured with three-ply tar paper between them, and an expansion joint, formed by a $\frac{1}{4}$ to $\frac{1}{2}$ in.-wide strip of asphaltic felt or bituminous expansion-joint material, is located at about every sixth block. Blocks laid in this manner will permit practically no water to get through their joints when expanded, but in cold weather they will contract somewhat and permit some leakage.

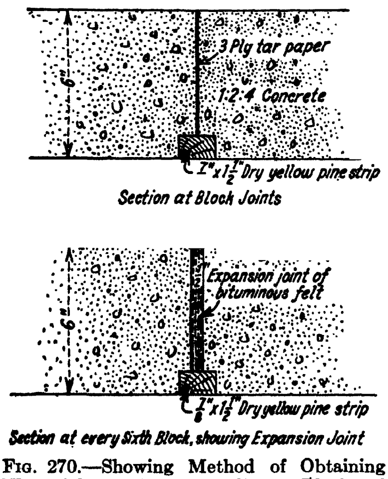


Fig. 270.—Showing Method of Obtaining Watertightness between Adjacent Blocks of Concrete Canal Lining.

An effective manner of securing water-tight joints between the blocks is indicated in Fig. 270. The wooden strips there shown make an effecive and economical expansion joint. When put in place, at the time when the concrete is poured, they are dry. They gradually take up water, swell, and become very tight in the concrete. When the blocks contract, the bottom surface of each block is still in contact with the top surface of the wooden strip, thus providing an effective water seal at all times. The objection to the use of these wooden strips is that if not kept continually wet they will decay. However, as in a power canal the strips will be kept wet all the time, they may be expected to last almost indefinitely. Sometimes metal strips, similar to the expansion joints shown in Fig. 147 for concrete core-walls, are used; but this is a refinement which is believed to be undesirable in most cases. Occasionally, where absolute water-tightness and absolute permanency are required, a reinforced concrete seat is provided at the block joints. A trench is cut in the subgrade at the location of the block joints, and a strip of concrete about 4 in. thick and 6 in. wide is poured and

[5] See Sec. 195.

finished to a smooth top surface. Tar paper is placed on this surface and also between the blocks, as indicated in Fig. 271. This concrete strip is reinforced lightly with wire mesh or expanded metal. Then, when the blocks expand or contract, they slide on the surface of the seat thus provided and water-tightness is maintained. With any of these types of concrete lining, the loss by seepage should be practically negligible.

Reinforced Concrete Linings.—In some cases, instead of using concrete slabs for lining the canal, a thinner layer of reinforced concrete is used. This is generally laid down monolithically without the use of expansion joints, the steel being depended on to take care of temperature and shrinkage stresses. It is claimed that a lining of this sort is tighter than one in which slabs are used. The steel probably causes the concrete to open up in a large number of minute cracks instead of in a few large cracks as would be the case if no steel were used. The thickness of concrete used is generally much less than that adopted when the lining is of slabs, and varies from $1\frac{1}{2}$ in. to 3 in. The steel reinforcement consists of wire cloth or expanded metal and is laid in position on the trimmed earth slopes and supported by means of chairs, small blocks of mortar, or strips of wood. The concrete is then poured, care being taken to retain the reinforcement in the concrete instead of allowing it to be pressed down against the earth by the weight of the fresh concrete. To insure this, it is often necessary, while the concrete is being poured, to take a hook, pull up the reinforcement, and joggle it so that some of the concrete gets underneath.

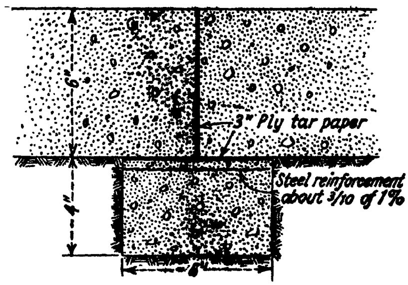


FIG. 271.—Showing Reinforced Concrete Seat at Block Joint.

When the slope of the concrete lining is 1 on 1 or flatter, it is better and more economical to place it without the use of forms, using a fairly stiff mixture and tamping it into place. The surface should be floated and troweled. A very smooth surface, having a low value for Kutter's n, can be obtained in this way. When the slopes are steeper than 1 on 1, forms are generally used; in this case a wetter mixture becomes necessary. The steel ratio generally used for such linings is about $\frac{3}{10}$ of 1 per cent or less. The concrete mixtures used vary from about 1 : 2 : 4 for the greater thicknesses of concrete to mortars without any stone or gravel for the $1\frac{1}{2}$-in. thicknesses. The mortars used for this purpose vary from 1 to $2\frac{1}{2}$ to as lean as 1 to 5. A mortar as lean as 1 to 5 is not recommended but has been used for this purpose. The real advantage of these thin reinforced concrete linings is that they are cheaper than the concrete blocks. An unreinforced concrete lining as thin as this would soon disintegrate in most climates. The addition of the steel holds it together, and an effective lining is generally secured except under conditions where the frost action is especially severe.

Gunite on Canal Slopes.—The use of "gunite" as a protection on the water slopes of earth dams has been mentioned in Sec. 112. It is just as applicable to the lining of a canal. After the slopes have been trimmed to final grade, building paper should be laid on them, if they are of earth. Then the mesh reinforcement is laid in place, being supported above the building paper by means of small pre-cast mortar blocks with pieces of wire, for fastening to the reinforcement, cast in them. Chairs or other suitable means may also be used, in lieu of the mortar blocks, for supporting the reinforcement in place. The cement gun is placed in position near the work and the gunite shot on, and, when built up to the required thickness, it is troweled smooth. This method gives a very strong and impervious surface. The thickness of gunite used for this purpose is usually from 1 to $1\frac{1}{2}$ in.

Clay Linings.—Clay is sometimes used as a lining for the bottom of canals; but such material is readily eroded, particularly during the emptying and filling of the canal. It should therefore be used with caution and well protected.

Slopes for Concrete Linings.—The slopes used for the concrete lining at the sides of the canal vary from 1 on 2 to 1 on $\frac{1}{2}$ for canals in earth. The selection of the slope used should be governed by the nature of the material on which the concrete lining must be placed. Thus a clayey material, which is likely to become partly saturated at times owing to the presence of ground water, would require that the concrete lining have a flat slope. On the other hand, if the material is sandy and free-draining, but with enough cohesion to have a steep angle of repose, the concrete lining may be placed on a relatively steep slope.

When canals through rock are lined with concrete, the sides are frequently made vertical.

Economics of Linings.—The cost of the lining should be credited with the sum of the capitalized annual value of the energy which would be lost through seepage, if the canal were not lined; the capitalized annual value of the energy saved by less friction loss in the lined canal; and the difference in cost of excavation between the unlined and lined canal (plus or minus).

Drainage of Concrete Linings.—Care should be taken that concrete linings are well drained if ground water is some distance above the bottom of the canal; otherwise the lining may be moved by the pressure of water on the underside when the canal is emptied. This is particularly likely to occur on the up-hill side of side-hill canals. In very cold climates the lining may be moved by frost action when the canal is empty, even if the water pressure is insufficient to cause damage. Effective drainage may be obtained either by loose stone or open tile drains under the canal floor, discharging at suitable intervals.

201. Canal Spillways.—With long canals, when side streams are allowed to discharge into the canal and when abnormal surges must be taken care of without a material rise in the water surface of the canal, spillways along the canal are used. These are generally located near the power house or in the forebay; but if the canal is long and there are a number of side streams discharging into it, it may be necessary to have several spillways distributed throughout its length. Such spillways are usually small overflow masonry

dams. Where larger quantities of water must be passed through a limited length of spillway crest, siphon spillways, as described in Sec. 144, and spillways surmounted by gates, as described in Sec. 137, are used.

202. Side-stream Tributaries.—When a canal is built along a side hill, the brooks and small streams discharging from the hills are sometimes permitted to discharge directly into the canal. Where the country is steep, such streams will usually have a high velocity during rains and will carry a large amount of silt and débris, which will often be deposited when the swiftly moving water of the brook impinges on the relatively slowly moving water of the canal. Under such conditions, the stream should be passed through a culvert or inverted siphon under the canal, and not allowed to enter the canal at all, as the deposit of débris at the mouth of the stream in the canal will either have to be periodically removed or it will form a submerged dam in the canal which will seriously affect the carrying capacity of the canal.

Where the canal embankment crosses relatively long and wide stream valleys, the same objection does not exist, for a considerable pond is thus created and the débris will be deposited near the upper end of the pond where the stream enters it. Eventually, of course, the pond may be silted up, and this factor should also be given consideration. This is not important, however, because of the length of time usually required for the pond to silt up to such an extent that deposits will begin to take place in the canal. Moreover, as the depth of water in such instances is usually very great, scouring sluices can be arranged in the base or sides of the canal to remove deposits.

Ponds formed by side streams are quite desirable, particularly at the lower end of the canal, because of their ability to act as regulators to supply the peak demand, allowing the canal above them to be designed for less than the peak flow.

203. Removal of Débris from Canals.—When the intake for the canal is at a diversion dam in a rapidly flowing river, the velocity of the water in the canal may, at times, be very much less than that in the river, and under such conditions deposits in the canal are likely to occur. Even in cases where the velocity in the canal is high enough to keep the sand in suspension, the resulting wear on the sealing rings of the turbines may be a serious matter, increasing clearances and decreasing efficiencies. To obviate this difficulty, the use of sand boxes and settling basins has been introduced. The problem is much the same as that presented in the design of settling basins and coagulation basins in filtration work, except that the permissible velocities are much higher. When there is a terminal pond or a large forebay, the velocity through them is so low that sand of a size to injure the turbines is deposited. If the forebay is utilized for this purpose, special means for removing the sand, akin to those described below, are frequently advisable.

It is usually considered that if the velocity is reduced to 0.5 ft. per second, practically all particles having a diameter greater than about 0.07 millimeter will be deposited, and that the particles remaining in suspension with this velocity will be too fine to cause appreciable wear on the turbine parts. In the design of a settling basin, great care must be used in distributing the velocity throughout the cross-section so that all parts of the cross-section have prac-

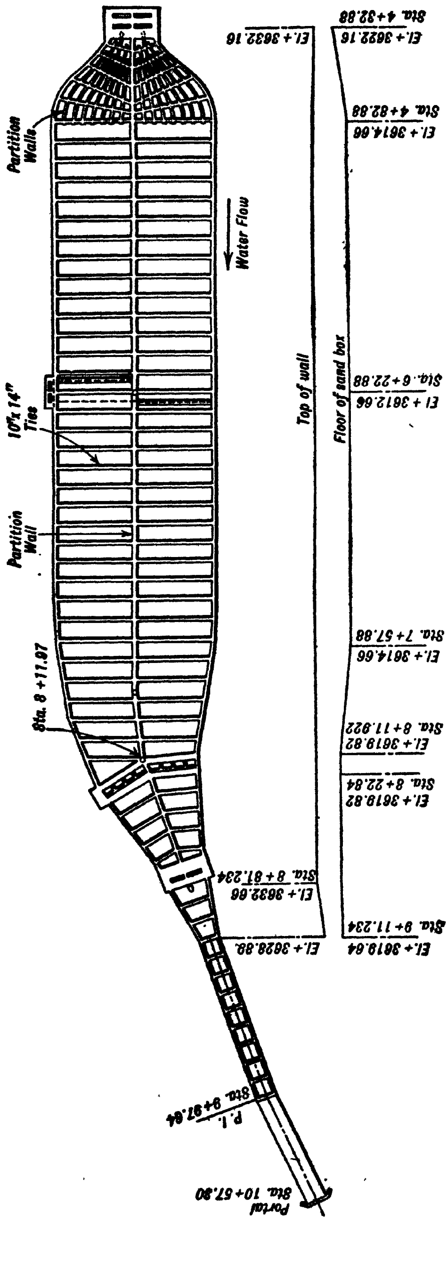


FIG. 272.—Settling Basin for Kern River No. 3 Plant of Southern California Edison Co. From Vol. 88, p. 616, Engineering News Record.

tically the same velocity. Otherwise, cross currents will be produced and will prevent the deposit of a large part of the sand.

This even distribution of velocity may be accomplished by means of a baffle wall or submerged weir, as is often done in settling basins for filtration plants. But such devices require a loss of head of several inches, which is, of course, very objectionable in a power canal. To overcome this objection, the Southern California Edison Company, in a settling basin for the Kern River No. 3 plant, used a flaring transition from the canal section to the settling basin, which was divided by a number of vertical walls. At the upstream end of each wall there was a hinged vane which could be moved one way or the other, and the quantity of water entering any particular section could thus be varied at will. By manipulating these vanes for any given condition of flow, a practically uniform velocity of flow was secured throughout the entire cross-

section of the settling basin. The general arrangement of this settling basin is shown in Fig. 272. The basin was 20 ft. deep, 60 ft. wide, and 400 ft. long, and the mean velocity through it was 0.5 ft. per second.

In order to draw off the sand which collects in the bottom of such a basin, mud valves with horizontal disks are often used. These valves are located in depressions in the floor of the basin and connect to a drainage system or conduit under the floor. In order to make them effective, it is necessary to install a considerable number of such mud valves, as each valve will only remove the sand from a limited area in its immediate vicinity. The settling basin in Fig. 272 was divided by means of a longitudinal vertical wall into two sections, so that when one was being cleaned of sand the other could remain in use. The bottom of each section was sloped toward the center, where there was a trench, connected with a 4-by-5-ft. sluice gate in the side of the basin. To remove the sand from one side of the settling basin, the gates at the upper and lower ends of the section were shut and this half section of the basin closed. The sluice gate was then opened and this half of the basin drained. The gates at the ends of the section were then cracked, and the sand on the floor washed out through the trench and sluice gate. Analysis of the sand on the floor of the basin showed that 45 per cent of it passed a 150-mesh sieve and 26 per cent a 200-mesh sieve.

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CHAPTER XIX

FLUMES

By William P. Creager and Joel D. Justin

205. General.—A general outline of the purpose and use of flumes in hydroelectric developments and a discussion of those features of flumes which are common to all types of conduits are given in Chapter XVII.

206. Types of Flumes.—A flume is simply a long trough of wood, concrete, or steel, supported on or above the ground surface and used to convey water. Steep, rocky hillsides, heavily wooded and with light soil cover, together with a necessity for low first cost, would favor the selection of a wooden flume made from lumber sawed on the ground. A similar topography, but without the timber and with the presence of suitable concrete materials, would favor the use of a reinforced concrete flume; or, if suitable concrete materials were not present, it might favor the use of steel flumes.

207. Economic Design.—For successful operation, the size of a flume for a given discharge may vary between wide limits; but there is usually one size which will make for greatest economy of design. The reader is referred to Sec. 87 for the general theory of economic design, and to Sec. 183 for its application to the special case of conduits. There is no practical limit to the velocity in flumes, and this is always determined by the principles of economic design unless the conditions are unusual. Usual velocities are 8 to 10 ft. per second.

208. Free-board.—The free-board, or elevation of the sides of the flume above calculated high-water surface, is almost invariably used to provide a factor of safety, to cover inaccuracies in the determination of the water surface.[1] The free-board should also be high enough to retain the surge due to a sudden shutting down of the turbine,[2] unless the overtopping of the sides under such conditions would cause no damage.

For very short flumes, it is usually economical and desirable to make the sides of the flume at least as high as ordinary high-water surface in the pond, to insure full head on the turbines when the water rises. If the flume cannot be overtopped without damage, the sides must be carried throughout to or above highest water surface in the pond, or control gates must be installed at the intake to limit the discharge into the flume.

Such control gates are invariably used for very long flumes, and the head due to high water is sacrificed for reduction in first cost of the flume. Under

[1] See Sec. 74.
[2] See Sec. 78.

such conditions, spillways should be provided at convenient places to discharge the excess flow in case the control gates are not kept in exact adjustment with the head on the intake.

209. Wooden Flumes.—Wooden flumes are used only where low first cost is of prime importance. Ten to twelve years for pine and fifteen years for redwood is the ultimate life of this type of flume unless it is creosoted, and maintenance charges are very high. Wooden flumes are also constantly in danger if brush fires are frequent. Short life and excessive cost of maintenance frequently preclude their use for economical permanent construction.

Rectangular Wooden Flumes.—The lagging used for lining the flume should be of good-quality timber, free from warps and knots, and should be equal to the best No. 1 grade. The thickness of this lagging is frequently 2 in. or more and is seldom less than $1\frac{1}{2}$ in. except for very small flumes. The side of the lagging that is to be in contact with the water should be planed. The additional expense is small, and the planing reduces the friction. It is, in fact, economical to size the lagging and plane both sides as this greatly facilitates handling and placing.

With butt lagging, (*a*) Fig. 273, the edges of the boards are beveled, and oakum, with pitch, is calked into the joints. Sometimes, instead of calking the joints, battens are nailed over them; but this practice is not particularly desirable as it decreases the effective section and increases the friction. The other forms of lagging shown in Fig. 273 are usually more effective and, of these forms, the butt lagging with splines (*e*) when properly constructed is the most desirable. Water-tightness at the end joints is sometimes secured by battening or by calking with oakum and pitch; but splines are more effective. The quality of the timber used for splines should be A1. It is frequently of white pine or, if the lagging itself is of soft wood, the splines are of oak or other hard wood. Galvanized iron splines have been effectively substituted for wooden splines in some cases, especially for the end joints.

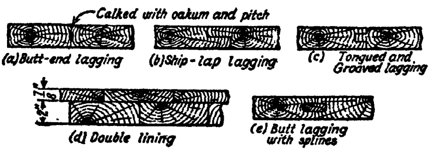


FIG. 273.—Various Types of Lagging for Timber Flumes.

The framework of the flume consists of vertical studs, to which the side lagging is nailed, and sills, to which the floor lagging is nailed. The sills usually extend beyond the studs, and near their ends they are dapped to take the thrust from an inclined brace which has its other end braced against the stud. The studs are usually also secured by tying across the top of the box, as indicated in Fig. 274, either by a rod or by a timber nailed to the studs. On large flumes there are sometimes intermediate ties, consisting of steel rods, between the studs and below the water surface. This practice, however, is not to be recommended, as the water causes the rods to vibrate and sometimes eventually causes crystallization and the breaking of the rods.

If the flume is on a bench cut in a hillside, the sills may rest directly on the ground or on blocking. Broken stone or coarse gravel under the sills, to facil-

itate drainage, prolongs their life. If the flume is on a trestle, the sills rest on longitudinal stringers which carry the load of the flume from trestle bent to trestle bent. The span between bents is usually from 12 to 16 ft. Fig. 275

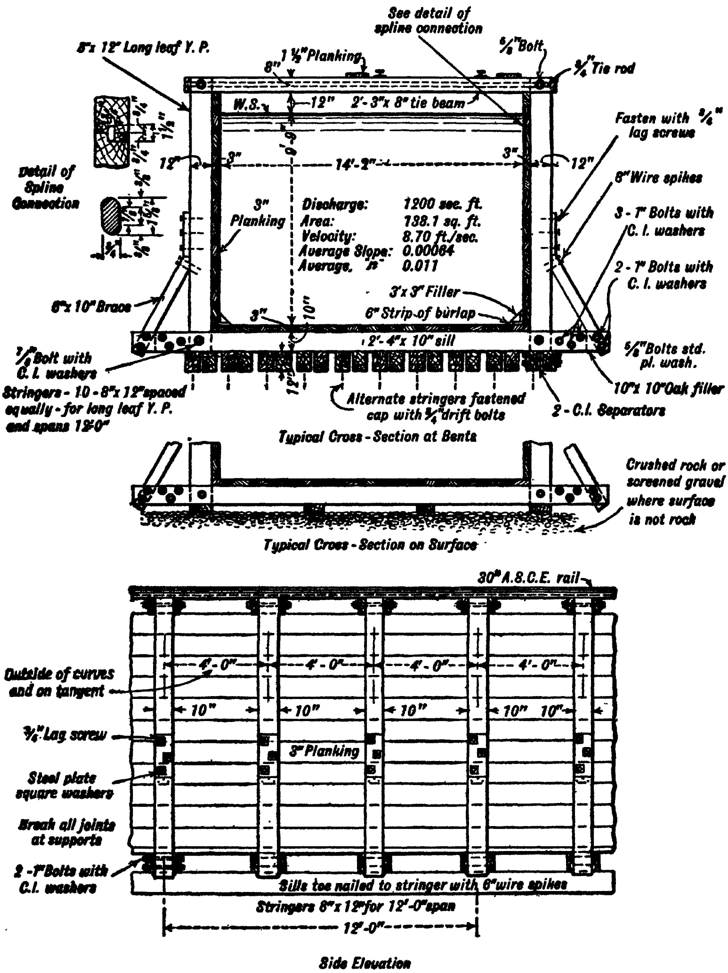


Fig. 274.—Flume of Ocoee No. 2 Development, Tennessee Power Co.

and Table XL give details of standard wooden flumes as used by the U. S. Reclamation Service, and Fig. 274 shows a timber flume of The Tennessee Power Company.

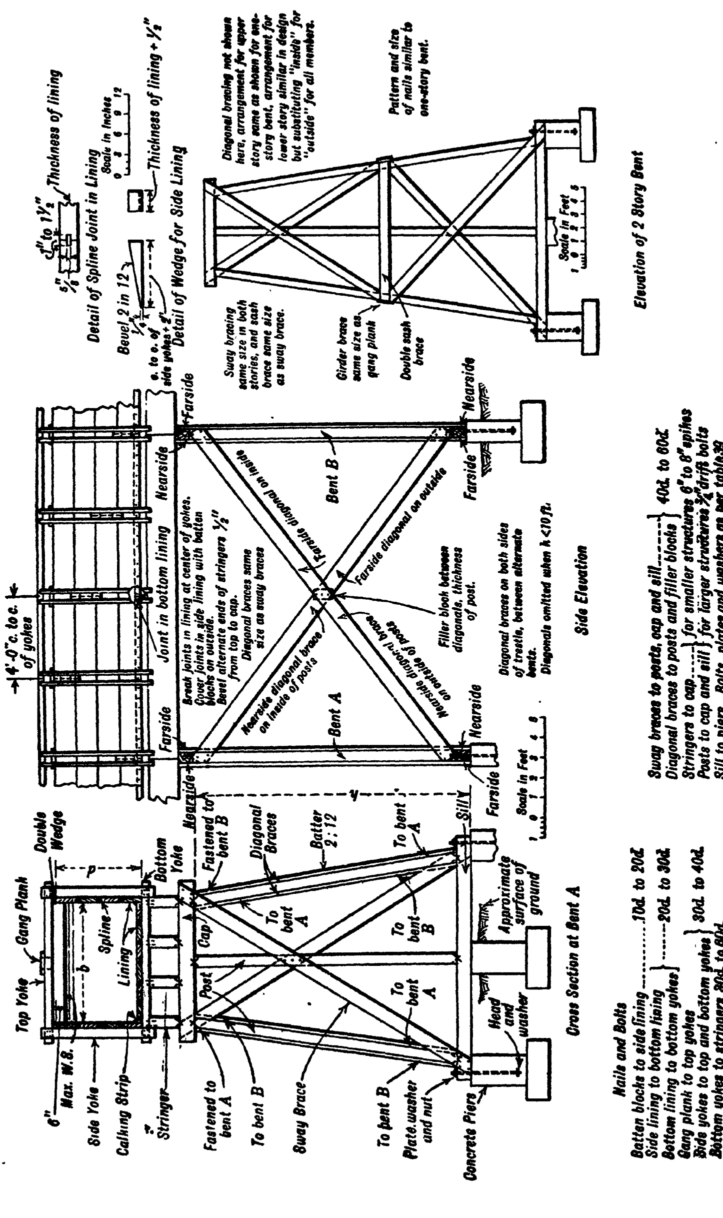


FIG. 275.—Standard Wooden Flumes. Sept., 1907. United States Reclamation Service.

TABLE

STANDARD WOODEN FLUMES—DIMENSIONS AND

United States Reclamation

		Flume										
					Stringers				Material			
						Dimensions in Inches			Lumber in Feet, B. M.			
$d \times b$ = Height of Flume × Inside Width, Feet	Span in Feet	Thickness of Lining, Inches	Dimensions of Side Yokes, Inches	Dimensions of Top and Bottom Yokes, Inches	Number	Class A	Class B	Class C	Class A	Class B	Class C	Nails in Pounds
3×3	12	2	2–2×4	4×4	3	4×12	4×10	4×10	480	460	460	18
3×3	16	2	2–2×4	4×4	3	6×12	5×12	4×12	730	680	630	22
3×4	12	2	2–2×4	4×4	3	5×12	4×12	3×12	550	510	480	18
3×4	16	2	2–2×4	4×4	3	6×14	6×12	5×12	820	770	720	22
3×5	12	2	2–2×4	4×4	4	4×12	3×12	4×10	600	560	560	21
3×5	16	2	2–2×4	4×4	4	5×14	6×12	5×12	910	910	850	25
3×6	12	2	2–2×4	4×4	4	5×12	4×12	3×12	680	630	580	22
3×6	16	2	2–2×4	4×4	4	6×14	5×14	6×12	1020	950	950	26
4×4	12	2½	2–2×6	4×4	3	6×12	5×12	4×12	750	710	680	20
4×4	16	2½	2–2×6	4×4	3	6×16	6×14	5×14	1080	1040	980	24
4×5	12	2½	2–2×6	4×4	4	5×12	4×12	4×12	820	770	770	23
4×5	16	2½	2–2×6	4×4	4	5×16	6×14	6×12	1200	1200	1150	27
4×6	12	2½	2–2×6	4×4	4	6×12	5×12	4×12	910	860	810	23
4×6	16	2½	2–2×6	4×4	4	6×16	5×16	5×14	1320	1240	1190	27
4×7	12	2½	2–2×6	4×4	5	6×12	5×12	4×12	1030	970	910	26
4×7	16	2½	2–2×6	4×4	5	6×16	6×14	5×14	1510	1430	1340	30
4×8	12	2½	2–2×6	4×4	5	5×14	5×12	4×12	1050	1000	950	26
4×8	16	2½	2–2×6	4×4	5	5×18	5×16	6×14	1520	1470	1470	30
5×5	12	2½	2–3×6	4×6	4	6×12	5×12	4×12	1010	960	910	26
5×5	16	2½	2–3×6	4×6	4	5×18	5×16	6×14	1410	1370	1370.	30
5×6	12	2½	2–3×6	4×6	4	6×14	6×12	5×12	1100	1050	1000	26
5×6	16	2½	2–3×6	4×6	4	6×18	6×16	5×16	1570	1500	1420	30
5×7	12	2½	2–3×6	4×6	5	5×14	6×12	5×12	1170	1170	1110	30
5×7	16	2½	2–3×6	4×6	5	6×18	6×16	6×14	1780	1700	1620	35
5×8	12	2½	2–3×6	4×6	6	5×14	5×12	4×12	1280	1230	1150	33
5×8	16	2½	2–3×6	4×6	6	5×18	5×16	6×14	1860	1800	1800	38
5×9	12	2½	2–3×6	4×6	6	6×14	4×14	5×12	1410	1260	1260	33
5×9	16	2½	2–3×6	4×6	6	6×18	5×18	5×16	2040	1900	1820	39
5×10	12	2½	2–3×6	4×6	7	5×14	4×14	5×12	1450	1370	1370	37
5×10	16	2½	2–3×6	4×6	7	6×18	6×16	5×16	2270	2150	2000	42

NOTE.—Three kinds of lumber have been assumed, classified as follows:

Class A: Nominal unit stress 800
Class B: Nominal unit stress 1000
Class C: Nominal unit stress 1200

Flume lining, yokes and posts have been considered only for class A.
Stringers have been considered for each class.
Caps and sills have been considered only for class C.
Quantities in table are for one flume span and one bent and include no allowance for culls, waste, etc.

XL

Quantities—Flumes and Trestle Bents

Service, September, 1907

Bents														
			Height 10 Feet					Maximum Height						
						Anchor Bolts					Anchor Bolts			
Dimensions of Caps and Sills, Inches	Number and Size of Posts, Inches	Maximum Height, Feet	Dimensions of Bracing, Inches	Lumber in Feet, B. M.	Nails in Pounds	Diameter, Inches	Weight, Pounds	Dimensions of Bracing, Inches	Lumber in Feet, B. M.	Nails in Pounds	Diameter, Inches	Weight, Pounds	Type of Concrete Piers See Drawing No. 3
4×8 6×6	2–4×4 2–6×6	10 15	2×6 2×6	130 160	8 9	7/8 7/8	25 25	2×6 3×6	130 250	8 10	7/8 7/8	25 25	2 2
6×6 6×6	3–6×6 3–6×6	15 15	2×6 2×6	200 200	9 9	3/4 3/4	17 17	3×6 3×6	300 310	10 10	7/8 7/8	25 25	2 2
6×6 6×6	3–6×6 3–6×6	15 15	2×8 2×8	220 230	9 9	5/8 5/8	12 12	3×8 3×8	350 360	10 10	3/4 3/4	17 17	2 2
6×6 6×8	3–6×6 3–6×6	15 15	2×8 2×8	250 260	9 9	5/8 5/8	12 12	3×8 3×8	360 390	10 10	3/4 3/4	17 17	2 2
6×6 6×6	3–6×6 3–6×6	15 15	2×6 2×6	200 210	9 9	7/8 7/8	25 25	3×6 3×6	300 310	10 10	1 1	32 32	3 3
6×6 6×8	3–6×6 3–6×6	15 15	2×8 2×8	230 250	9 9	3/4 3/4	17 17	3×8 3×8	350 380	10 10	7/8 7/8	25 25	3 3
6×8 6×8	3–6×6 3–6×6	15 15	2×8 2×8	260 270	9 9	3/4 3/4	17 17	3×8 3×8	380 390	10 10	7/8 7/8	25 25	3 3
6×8 6×10	3–6×6 2–6×6 1–6×8	15 15	2×8 2×8	270 310	9 9	5/8 5/8	12 12	3×8 3×8	390 440	10 10	3/4 3/4	17 17	3 3
6×10 8×10	3–6×6 3–8×8	15 20	3×10 3×10	370 500	9 9	5/8 5/8	12 12	3×10 3×10	470 760	10 10	3/4 3/4	17 17	3 3
6×8 6×8	3–6×6 3–6×6	15 15	2×8 2×8	240 250	9 9	1 1	32 32	3×8 3×8	370 380	10 10	1 1	32 32	4 4
6×8 8×8	3–6×6 3–8×8	15 20	2×8 2×8	250 360	9 9	7/8 7/8	25 25	3×8 3×8	380 650	10 10	1 1	32 32	4 4
6×8 8×8	2–6×6 1–6×8 3–8×8	15 20	2×8 2×8	280 370	10 10	3/4 3/4	17 17	3×8 3×8	390 660	10 12	7/8 7/8	25 25	4 4
8×10 8×10	3–8×8 3–8×8	20 20	3×10 3×10	470 500	10 10	5/8 5/8	12 12	3×10 3×10	750 760	12 12	7/8 7/8	25 25	4 4
8×10 8×12	3–8×8 2–8×8 1–8×10	20 20	3×10 3×10	500 570	10 10	5/8 5/8	12 12	3×10 3×10	770 840	12 12	7/8 7/8	25 25	4 4
8×10 8×12	3–8×8 2–8×8 1–8×10	20 20	3×10 3×10	520 590	10 10	5/8 5/8	12 12	3×10 3×10	780 860	12 12	3/4 3/4	17 17	4 4

Anchor bolts shall be imbedded in concrete piers about 24 inches and provided with washers at imbedded end and plates on sills as follows:

Dia. Bolt	Dia. Washer	Size of Plate
1/2″	2″	4″×4″×3/8″
5/8″	2″	4″×4″×1/2″
3/4″	3″	4″×6″×1/2″
7/8″	3″	6″×6″×5/8″
1″	4″	6″×7″×5/8″

Wood-stave Flumes.—Semi-circular wood-stave flumes are used quite extensively in the West for the conveyance of water. The details of such

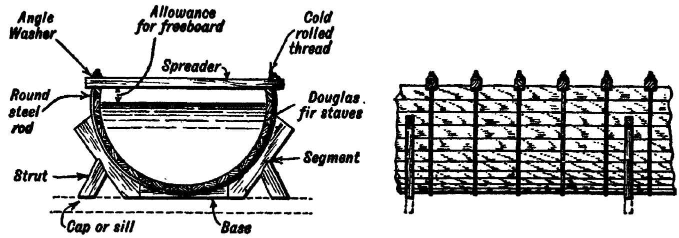


FIG. 276.—Semi-circular Wood Stave Flume as Made by Continental Pipe Mfg. Co.

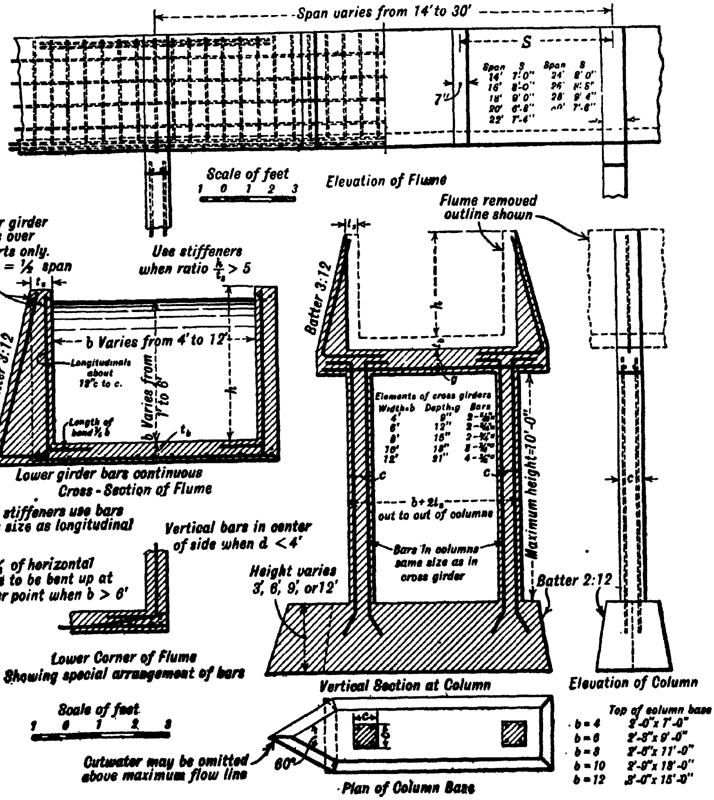


FIG. 277.—Standard Concrete Flumes, 1907, United States Reclamation Service.

flumes and the method of construction are similar to those used for wood-stave pipe lines (see Sec. 232). Staves and cradles should in all cases be

creosoted to promote long life. As indicated in Fig. 276, there is a horizontal yoke, or spreader, to which the steel rods that confine the staves are attached. The spacing of the cradles is usually 8 ft. center to center.

210. Reinforced Concrete Flumes.—Reinforced concrete flumes are almost invariably the most expensive type, but are the most permanent and satisfactory. They are made in many different forms, each a product of the concrete designer's methods. They are usually accompanied by reinforced concrete trestles.

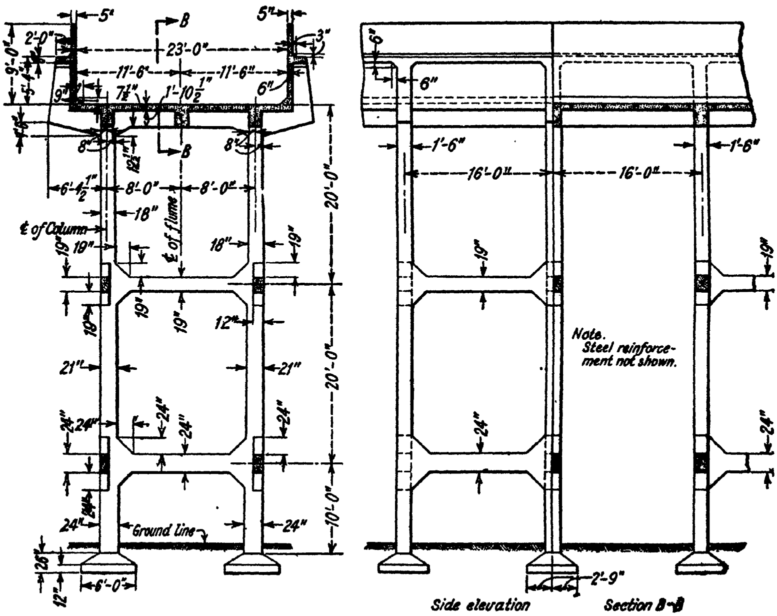


FIG. 278.—Cantilever Type of Concrete Flume on Trestles.

Figure 277 and Table XLI give details and quantities for typical reinforced concrete flumes as designed and used by the U. S. Reclamation Service. In this type of flume the side walls act as girders to support the flume between trestle bents. The flume shown in Fig. 278 is carried on separate beams which span between the top members of the bent. This top member is designed as a cantilever to support the beams carrying the load from the sides of the flume.

211. Steel Flumes.—Steel flumes are constructed in several different forms. Some have vertical sides with semi-circular or half-oval shaped bottoms and some are semi-circular. The vertical sides are often designed as plate girders to carry the load of the flume and its contents between supports and are also designed to take the water pressure. Sometimes the flumes are tied across the top with steel rods or angles, and sometimes the sides are designed to act as cantilevers.

TABLE XLI

Standard Concrete Flumes—Dimensions and Quantities—Spans, 14, 16, 18, 20 and 22 Feet

United States Reclamation Service. May, 1907

Span Feet	$d \times b$ = Depth of Water × Inside Width, Feet	h = Inside Height	t_s = Side Thickness, Inches	Size and Spacing of Transverse Bars in Side, Inches	t_b = Bottom Thickness, Inches	Size and Spacing of Transverse Bars in Bottom, Inches	Number and Size of Longitudinal Bars, Inches	Number and Size of Girder Bars, Inches		Cubic Yards of Concrete in Span Length	Pounds of Plain Steel Bars in Span Length	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners		
								Upper	Lower			Size of Column Inches	Cubic Yards of Concrete	Pounds of Plain Steel Bars
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
14	2×4	2′ 6″	7	3/8-18	7	3/8-18	8-3/8	1-5/8	2-5/8	3.1	210	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	1-5/8	2-5/8	4.4	250	9	0.7	180
	2×6	2′ 6″	7	3/8-18	7	3/8-9	10-3/8	4-3/8	2-5/8	3.7	290	9	0.8	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	2-1/2	2-5/8	5.0	330	9	0.9	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/8	2-1/2	2-5/8	5.0	410	9	1.0	260
	2×8	2′ 6″	7	3/8-18	7	1/2-10	12-3/8	3-1/2	2-3/4	4.2	400	9	0.9	290
	3×8	3′ 6″	9	3/8-18	7	1/2-8	16-3/8	3-1/2	2-3/4	5.6	470	9	1.0	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-1/2	3-1/2	2-3/4	6.1	630	10	1.2	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-1/2	2-5/8	2-3/4	7.2	790	12	1.6	300
	3×10	3′ 6″	9	3/8-18	9	5/8-10	14-5/8	2-5/8	3-5/8	7.2	730	10	1.4	290
	4×10	4′ 6″	7	3/8-9	9	3/4-12	14-5/8	2-5/8	3-5/8	7.2	880	12	1.8	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	2-5/8	3-5/8	8.4	1050	12	1.9	290
	3×12	3′ 6″	9	3/8-18	10	3/4-12	17-5/8	3-5/8	4-5/8	8.6	950	12	2.0	300
	4×12	4′ 6″	7	3/8-9	12	3/4-12	19-5/8	3-5/8	4-5/8	9.7	1080	15	2.9	330
	5×12	5′ 6″	7	1/2-10	12	3/4-9	20-5/8	3-5/8	4-5/8	10.5	1330	15	3.1	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	3-5/8	4-5/8	12.4	1530	15	3.2	340
16	2×4	2′ 6″	7	3/8-18	7	3/8-18	8-3/8	4-3/8	2-5/8	3.6	240	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	1-3/4	2-3/4	5.0	330	9	0.7	180
	2×6	2′ 6″	7	3/8-18	7	3/8-9	10-3/8	5-3/8	2-3/4	4.2	360	9	0.8	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	3-1/2	2-3/4	5.7	430	9	0.9	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/8	3-1/2	2-3/4	5.8	520	10	1.3	260
	2×8	2′ 6″	7	3/8-18	7	1/2-10	12-3/8	1-7/8	2-7/8	4.8	500	9	0.9	290
	3×8	3′ 6″	9	3/8-18	7	1/2-8	16-3/8	1-7/8	2-7/8	6.4	580	10	1.1	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-1/2	1-7/8	2-7/8	6.8	770	12	1.5	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-1/2	2-5/8	3-5/8	8.2	880	12	1.6	300
	3×10	3′ 6″	9	3/8-18	9	5/8-10	14-5/8	2-3/4	3-3/4	8.2	910	12	1.7	290
	4×10	4′ 6″	7	3/8-9	9	3/4-12	14-5/8	3-5/8	4-5/8	8.2	1030	12	1.8	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	3-5/8	4-5/8	9.5	1230	15	2.7	290
	3×12	3′ 6″	9	3/8-18	10	3/4-12	17-5/8	3-5/8	5-5/8	9.8	1130	12	2.0	300
	4×12	4′ 6″	7	3/8-9	12	3/4-12	19-5/8	3-5/8	5-5/8	11.0	1250	15	2.9	330
	5×12	5′ 6″	7	1/2-10	12	3/4-9	20-5/8	2-3/4	3-3/4	11.9	1510	15	3.1	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	3-5/8	5-5/8	14.1	1780	18	4.1	340

TABLE XLI—*Continued*

Span Feet	$d \times b$ = Depth of Water × Inside Width, Feet	h = Inside Height	t_s = Side Thickness, Inches	Size and Spacing of Transverse Bars in Side, Inches	t_b = Bottom Thickness, Inches	Size and Spacing of Transverse Bars in Bottom, Inches	Number and Size of Longitudinal Bars, Inches	Number and Size of Girder Bars, Inches		Cubic Yards of Concrete in Span Length	Pounds of Plain Steel Bars in Span Length	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners		
								Upper	Lower			Size of Column Inches	Cubic Yards of Concrete	Pounds of Plain Steel Bars
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
18	2×4	2′ 6″	7	3/8-18	7	3/8-18	8-3/8	1-3/4	2-3/4	4.0	320	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	1-3/4	2-3/4	5.6	370	9	0.7	180
	2×6	2′ 6″	7	3/8-18	7	3/8-9	10-3/8	1-7/8	2-7/8	4.7	470	9	0.8	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	1-7/8	2-7/8	6.4	530	10	1.1	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/8	1-7/8	2-7/8	6.5	640	10	1.3	260
	2×8	2′ 6″	7	3/8-18	7	1/2-10	12-3/8	2-3/4	3-3/4	5.4	610	9	0.9	290
	3×8	3′ 6″	9	3/8-18	7	1/2-8	16-3/8	2-3/4	3-3/4	7.2	700	12	1.4	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-1/2	3-5/8	4-5/8	7.8	900	12	1.5	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-1/2	3-5/8	4-5/8	9.2	1080	15	2.3	300
	3×10	3′ 6″	9	3/8-18	9	5/8-10	14-5/8	3-5/8	5-5/8	9.3	1060	12	1.7	290
	4×10	4′ 6″	7	3/8-9	9	3/4-12	14-5/8	3-5/8	5-5/8	9.2	1230	15	2.6	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	3-5/8	5-5/8	10.7	1450	15	2.7	290
	3×12	3′ 6″	9	3/8-18	10	3/4-12	17-5/8	4-5/8	6-5/8	11.0	1350	15	2.7	330
	4×12	4′ 6″	7	3/8-9	12	3/4-12	19-5/8	3-3/4	4-3/4	12.4	1520	15	2.9	330
	5×12	5′ 6″	7	1/2-10	12	3/4-9	20-5/8	3-3/4	4-3/4	13.4	1820	18	3.1	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	3-3/4	4-3/4	15.8	2100	18	4.1	340
20	2×4	2′ 6″	7	3/8-18	7	3/8-18	8-3/8	1-7/8	2-7/8	4.4	420	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	1-7/8	2-7/8	6.3	480	9	0.7	180
	2×6	2′ 6″	7	3/8-18	7	3/8-9	10-3/8	2-3/4	3-3/4	5.3	560	9	0.8	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	2-3/4	3-3/4	7.1	640	10	1.1	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/8	3-5/8	4-5/8	7.3	750	12	1.5	260
	2×8	2′ 6″	10	3/8-18	7	1/2-10	14-3/8	3-3/4	4-3/4	7.3	810	10	1.1	290
	3×8	3′ 6″	9	3/8-18	7	1/2-8	16-3/8	3-3/4	4-3/4	8.0	900	12	1.4	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-1/2	3-5/8	5-5/8	8.8	1060	12	1.5	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-1/2	3-5/8	5-5/8	10.4	1280	15	2.3	300
	3×10	3′ 6″	9	3/8-18	9	5/8-10	14-5/8	3-3/4	5-3/4	10.3	1330	12	1.7	290
	4×10	4′ 6″	7	3/8-9	9	3/4-12	14-5/8	3-3/4	4-3/4	10.3	1460	15	2.6	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	3-3/4	4-3/4	12.0	1720	15	2.7	290
	3×12	3′ 6″	9	3/8-18	10	3/4-12	17-5/8	2-1	3-1	12.2	1620	15	2.7	330
	4×12	4′ 6″	7	3/8-9	12	3/4-12	19-5/8	3-3/4	5-3/4	13.9	1770	15	2.9	330
	5×12	5′ 6″	7	1/2-10	12	3/4-9	20-5/8	3-3/4	5-3/4	15.0	2130	18	3.9	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	3-3/4	5-3/4	17.8	2430	18	4.1	340

TABLE XLI—*Continued*

Span Feet	$d \times b$ = Depth of Water × Inside Width, Feet	h = Inside Height	t_s = Side Thickness, Inches	Size and Spacing of Transverse Bars in Side, Inches	t_b = Bottom Thickness, Inches	Size and Spacing of Transverse Bars in Bottom, Inches	Number and Size of Longitudinal Bars, Inches	Number and Size of Girder Bars, Inches: Upper	Number and Size of Girder Bars, Inches: Lower	Cubic Yards of Concrete in Span Length	Pounds of Plain Steel Bars in Span Length	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners: Size of Column Inches	Cubic Yards of Concrete	Pounds of Plain Steel Bars
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
22	2×4	2′ 6″	7	3/8-18	7	3/8-18	8-3/8	3-5/8	4-5/8	4.9	500	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	3-5/8	4-5/8	6.9	560	9	0.7	180
	2×6	2′ 6″	10	3/8-18	7	3/8-9	12-3/8	3-3/4	4-3/4	7.0	770	10	1.0	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	1-1	2-1	7.8	740	12	1.3	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/8	3-5/8	5-5/8	8.0	890	12	1.5	260
	2×8	2′ 6″	12	3/8-18	7	1/2-10	16-3/8	2-1	3-1	8.8	1060	12	1.4	290
	3×8	3′ 6″	9	3/8-18	7	1/2-8	16-3/8	3-3/4	5-3/4	8.8	1080	12	1.4	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-1/2	2-7/8	3-7/8	9.6	1230	15	2.2	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-1/2	2-7/8	3-7/8	11.3	1480	15	2.3	300
	3×10	3′ 6″	9	3/8-18	9	5/8-10	14-5/8	2-1	3-1	11.3	1520	15	2.4	290
	4×10	4′ 6″	7	3/8-9	9	3/4-12	14-5/8	3-3/4	5-3/4	11.3	1680	15	2.6	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	3-3/4	5-3/4	13.2	1970	15	2.7	290
	3×12	3′ 6″	12	3/8-18	10	3/4-12	19-5/8	3-7/8	5-7/8	15.2	2020	15	2.7	330
	4×12	4′ 6″	8	3/8-12	12	3/4-12	20-5/8	4-3/4	6-3/4	16.1	2050	18	3.7	330
	5×12	5′ 6″	7	1/2-10	12	3/4-9	20-5/8	4-3/4	6-3/4	16.5	2460	18	3.9	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	4-3/4	6-3/4	19.6	2800	18	4.1	340
24	2×4	2′ 6″	9	3/8-18	7	3/8-18	10-3/8	3-3/4	4-3/4	6.2	730	9	0.7	180
	3×4	3′ 6″	9	3/8-18	7	3/8-18	12-3/8	3-5/8	5-5/8	7.6	660	10	0.8	180
	2×6	2′ 6″	12	3/8-18	7	3/8-9	14-3/8	4-3/4	5-3/4	8.6	1010	10	1.0	250
	3×6	3′ 6″	9	3/8-18	7	1/2-12	14-3/8	3-3/4	5-3/4	8.5	1000	12	1.3	260
	4×6	4′ 6″	7	3/8-9	7	1/2-10	14-3/4	3-3/4	4-3/4	8.7	1060	12	1.5	260
	2×8	3′ 0″	12	3/8-18	7	1/2-9	17-3/8	4-3/4	6-3/4	10.5	1250	12	1.4	290
	3×8	3′ 6″	10	3/8-18	7	1/2-8	16-3/8	3-7/8	4-7/8	10.2	1250	15	2.1	290
	4×8	4′ 6″	7	3/8-9	8	1/2-7	14-3/8	3-3/4	5-3/4	10.4	1320	15	2.2	300
	5×8	5′ 6″	7	1/2-10	9	5/8-10	16-3/8	3-3/4	5-3/4	12.3	1570	15	2.3	300
	3×10	3′ 6″	12	3/8-18	9	5/8-10	16-5/8	3-7/8	5-7/8	14.2	1900	15	2.4	290
	4×10	4′ 6″	8	3/8-12	9	3/4-12	15-5/8	4-3/4	6-3/4	13.1	1980	15	2.6	290
	5×10	5′ 6″	7	1/2-10	10	3/4-11	16-5/8	4-3/4	6-3/4	14.4	2260	18	3.5	310
	3×12	3′ 9″	12	3/8-18	10	3/4-12	20-5/8	4-7/8	6-7/8	16.9	2420	18	3.5	330
	4×12	4′ 6″	10	3/8-18	12	3/4-12	22-5/8	2-1 1/4	3-1 1/4	19.1	2550	18	3.7	330
	5×12	5′ 6″	8	1/2-12	12	3/4-9	21-5/8	4-7/8	5-7/8	18.9	2800	18	3.9	340
	6×12	6′ 6″	9	1/2-9	12	3/4-8	24-5/8	4-7/8	5-7/8	21.2	3190	21	5.1	360

TABLE XLI—*Continued*

Span Feet	$d \times b$ = Depth of Water × Inside Width, Feet	h = Inside Height	t_s = Side Thickness, Inches	Size and Spacing of Transverse Bars in Side, Inches	t_b = Bottom Thickness, Inches	Size and Spacing of Transverse Bars in Bottom, Inches	Number and Size of Longitudinal Bars, Inches	Number and Size of Girder Bars, Inches: Upper	Number and Size of Girder Bars, Inches: Lower	Cubic Yards of Concrete in Span Length	Pounds of Plain Steel Bars in Span Length	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners: Size of Column Inches	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners: Cubic Yards of Concrete	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners: Pounds of Plain Steel Bars
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
26	2×4	2′ 6″	12	⅜-18	7	⅜-18	12-⅜	3-¾	5-¾	8.2	920	10	0.8	180
	3×4	3′ 6″	9	⅜-18	7	⅜-18	12-⅜	3-¾	4-¾	8.2	830	10	0.8	180
	2×6	3′ 0″	12	⅜-18	7	⅜-9	15-⅜	4-¾	6-¾	10.2	1220	12	1.2	250
	3×6	3′ 6″	9	⅜-18	7	½-12	14-⅜	2-1	3-1	9.2	1140	12	1.3	260
	4×6	4′ 6″	7	⅜-9	7	½-10	14-⅜	3-¾	5-¾	9.4	1240	15	1.7	260
	2×8	Maximum height of side will not give sufficient strength as girder												
	3×8	3′ 6″	12	⅜-18	7	½-8	18-⅜	3-⅞	5-⅞	12.3	1540	15	2.1	290
	4×8	4′ 6″	8	⅜-12	8	½-7	15-½	4-¾	6-¾	12.1	1680	15	2.2	300
	5×8	5′ 6″	7	½-10	9	⅝-10	16-½	4-¾	6-¾	13.3	1950	15	2.3	300
	3×10	4′ 0″	12	⅜-18	9	⅝-10	17-⅝	2-1¼	3-1¼	16.4	2330	18	3.2	310
	4×10	4′ 6″	10	⅜-18	9	¾-12	17-⅝	2-1¼	3-1¼	15.9	2460	18	3.4	310
	5×10	5′ 6″	8	½-12	10	¾-11	17-⅝	3-1	4-1	16.6	2620	18	3.5	310
	3×12	Maximum height of side will not give sufficient strength as girder												
	4×12	4′ 6″	12	⅜-18	12	¾-12	24-⅝	4-1	6-1	22.4	3170	18	3.7	330
	5×12	5′ 6″	9	½-15	12	¾-9	23-⅝	3-1	5-1	21.4	3240	18	3.9	340
	6×12	6′ 6″	9	½-9	12	¾-8	24-⅝	2-1¼	3-1¼	23.0	3600	21	5.1	360
28	2×4	2′ 9″	12	⅜-18	7	⅜-18	13-⅜	3-⅞	4-⅞	9.3	1110	10	0.8	180
	3×4	3′ 6″	9	⅜-18	7	⅜-18	12-⅜	3-¾	5-¾	8.8	1000	12	1.1	180
	2×6	Maximum height of side will not give sufficient strength as girder												
	3×6	3′ 6″	12	⅜-18	7	½-12	16-⅜	3-⅞	5-⅞	12.0	1450	15	1.5	260
	4×6	4′ 6″	8	⅜-12	7	½-10	16-⅜	4-¾	6-¾	10.9	1470	15	1.7	260
	2×8	Maximum height of side will not give sufficient strength as girder												
	3×8	4′ 0″	12	⅜-18	8	½-8	21-⅜	2-1¼	3-1¼	15.2	1950	15	2.1	290
	4×8	4′ 6″	10	⅜-18	8	½-7	17-½	2-1¼	3-1¼	14.8	2140	15	2.2	300
	5×8	5′ 6″	8	½-10	9	⅝-10	17-½	3-⅞	5-⅞	15.3	2260	18	3.1	320
	3×10	Maximum height of side will not give sufficient strength as girder												
	4×10	4′ 6″	12	⅜-18	9	¾-12	18-⅝	4-1	6-1	18.9	3050	18	3.4	310
	5×10	5′ 6″	9	½-15	10	¾-11	18-⅝	2-1¼	3-1¼	19.0	2930	18	3.5	310
	3×12	Maximum height of side will not give sufficient strength as girder												
	4×12	5′ 0″	12	⅜-15	12	¾-10	25-⅝	4-1	6-1	25.3	3600	21	4.7	350
	5×12	5′ 6″	12	½-18	12	¾-9	26-⅝	4-1	6-1	26.5	3850	21	4.9	360
	6×12	6′ 6″	9	½-9	12	¾-8	24-⅝	4-1	6-1	24.7	4200	21	5.1	360

TABLE XLI—*Continued*

Span Feet	$d \times b$ = Depth of Water × Inside Width, Feet	h = Inside Height	t_s = Side Thickness, Inches	Size and Spacing of Transverse Bars in Side, Inches	t_b = Bottom Thickness, Inches	Size and Spacing of Transverse Bars in Bottom, Inches	Number and Size of Longitudinal Bars, Inches	Number and Size of Girder Bars, Inches		Cubic Yards of Concrete in Span Length	Pounds of Plain Steel Bars in Span Length	2 Columns, 10 Feet High, Including Cross Girder and Stiffeners		
								Upper	Lower			Size of Column Inches	Cubic Yards of Concrete	Pounds of Plain Steel Bars
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
30	2×4	3′ 0″	12	$\frac{3}{8}$–18	7	$\frac{3}{8}$–18	14–$\frac{3}{8}$	4–$\frac{3}{4}$	6–$\frac{3}{4}$	10.6	1270	12	1.1	180
	3×4	3′ 6″	10	$\frac{3}{8}$–18	7	$\frac{3}{8}$–18	13–$\frac{3}{8}$	3–$\frac{7}{8}$	4–$\frac{7}{8}$	10.2	1200	12	1.1	180
	2×6	Maximum height of side will not give sufficient strength as girder												
	3×6	3′ 9″	12	$\frac{3}{8}$–18	7	$\frac{1}{2}$–12	17–$\frac{3}{8}$	2–1$\frac{1}{4}$	3–1$\frac{1}{4}$	13.6	1810	15	1.5	260
	4×6	4′ 6″	10	$\frac{3}{8}$–18	7	$\frac{1}{2}$–10	18–$\frac{3}{8}$	3–1	4–1	13.7	1780	15	1.7	260
	2×8	Maximum height of side will not give sufficient strength as girder												
	3×8	Maximum height of side will not give sufficient strength as girder												
	4×8	4′ 6″	12	$\frac{3}{8}$–18	8	$\frac{1}{2}$–7	19–$\frac{1}{2}$	3–1	5–1	17.0	2420	18	3.0	320
	5×8	5′ 6″	9	$\frac{1}{2}$–15	9	$\frac{5}{8}$–10	19–$\frac{1}{2}$	4–$\frac{7}{8}$	6–$\frac{7}{8}$	17.7	2600	18	3.1	320
	3×10	Maximum height of side will not give sufficient strength as girder												
	4×10	5′ 0″	12	$\frac{3}{8}$–15	10	$\frac{3}{4}$–12	20–$\frac{5}{8}$	4–1	6–1	22.7	3440	18	3.4	310
	5×10	5′ 6″	12	$\frac{1}{2}$–18	10	$\frac{3}{4}$–11	22–$\frac{5}{8}$	4–1	6–1	24.1	3600	21	4.4	360
	3×12	Maximum height of side will not give sufficient strength as girder												
	4×12	Maximum height of side will not give sufficient strength as girder												
	5×12	5′ 9″	12	$\frac{1}{2}$–18	12	$\frac{3}{4}$–9	26–$\frac{5}{8}$	2–1$\frac{1}{2}$	3–1$\frac{1}{2}$	29.1	4380	21	4.9	360
	6×12	6′ 6″	12	$\frac{1}{2}$–12	12	$\frac{3}{4}$–8	28–$\frac{5}{8}$	2–1$\frac{1}{2}$	3–1$\frac{1}{2}$	31.2	4820	24	6.3	380

Semi-circular steel flumes have been developed in a manner which promotes ease and speed of construction. These are several patented types of

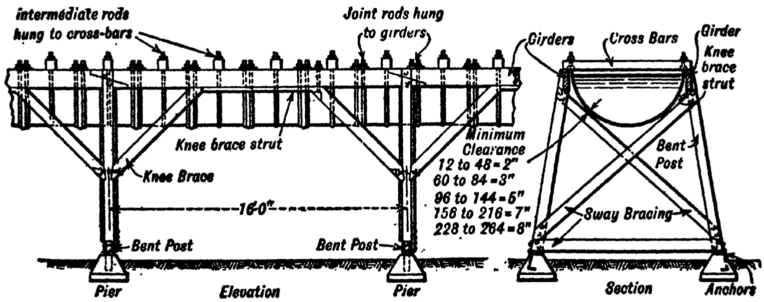


FIG. 279.—Details of Typical Semi-circular Steel Flume. California Corregated Culvert Co., Berkeley, Calif.

semi-circular steel flumes on the market. They are composed of sheets of steel which are sometimes galvanized. They are manufactured in sizes running from 1 ft. to 20 ft. in diameter.

The upstream and downstream edges of each sheet are crimped to form a groove. In assembling, the sheets are placed so that the grooves of adjacent sections lap together. A rod or light curved channel, *a*, Fig. 280, then fits into the groove on the inside of the flume and either one or two rods, *b*, fit around the flume on the outside of the groove. The outside rods are threaded at their ends and pass through the stringer, in many cases. At the end of each

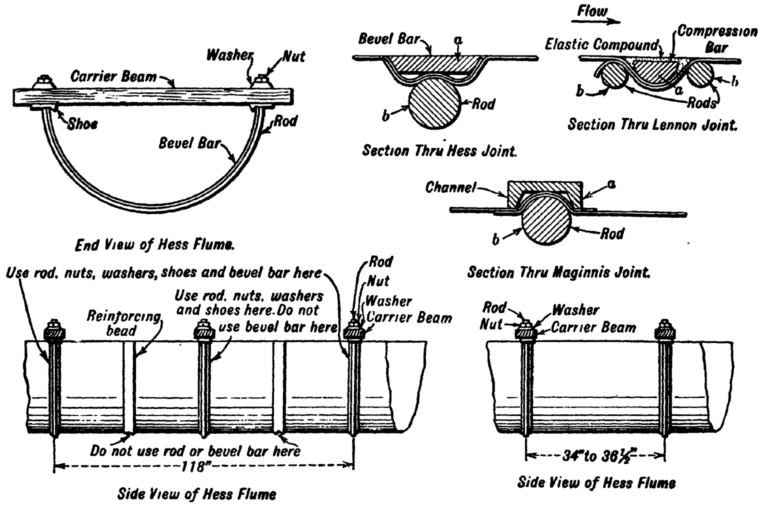


FIG. 280.—Details of Three Joints for Semi-circular Steel Flumes.*

* From a catalogue of the R. Hardesty Mfg. Co., Denver, Colo.

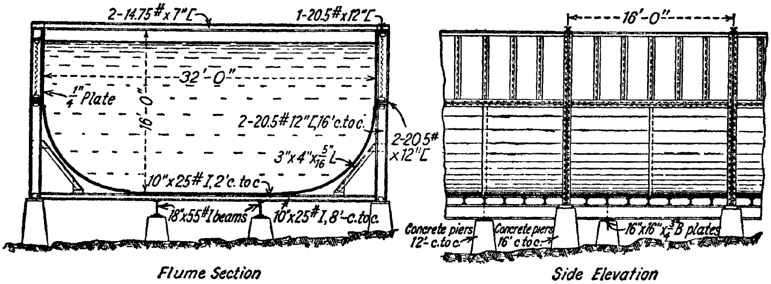


FIG. 281.—Steel Flume at Raymondville. Eaton and Brownell, Engineers.

of these rods there is a nut and washer. By screwing up the nut the sheets are pressed tightly together at the joint. In some makes of these flumes, the rods are secured to hanger plates which in turn bear on the stringer. Fig. 279 shows typical details of a semi-circular steel flume.

Some quite large flumes have been built of structural steel shapes and plates with riveted connections. Included in this type is the flume shown in Fig. 281. This flume is approximately 1600 ft. long and there is no expan-

sion joint in this entire length. The flume has been in service about twenty years and there has been some buckling of the plates, due to expansion, but this has never occasioned any extensive repairs.

212. Bibliography.—

1. Economic Construction of Flumes for Hydro-electric Plants. Trans. Am. Soc. C. E. Vol. LXXIX, p. 1010 (1915),
2. Flume without Side Braces. Eng. News, Vol. 69, p. 1041.
3. Design of Reinforced Concrete Flumes. Eng. News, Vol. 77, p. 66.
4. Concrete and Gunite Flumes on the King Hill Project, by Walter Ward. Eng. News-Record, Vol. 87, p. 926.

CHAPTER XX

STEEL PIPE

BY WILLIAM P. CREAGER AND JOEL D. JUSTIN

213. General.—A general outline of the purpose and use of pipes in hydro-electric developments and a discussion of those features of pipes which are common to all types of conduits are given in Chapter XVII. The pipe between the forebay and the surge tank is commonly called the "pipe line," and that between the surge tank and the turbines is called the "penstock."

214. Types of Pipes.—The two types of steel pipe used in hydro-electric developments differ only in the kind of joints, i.e., "riveted pipe" and "welded pipe." Riveted pipe is the more common, particularly for low and medium heads; but the use of welded pipe in large sizes has been increasing rapidly of late, especially in the West.

The advantages claimed for welded pipe are that it is lighter for the same strength than riveted pipe; that its carrying capacity is greater, for the same diameter, than that of riveted pipe; and that it is cheaper to install than riveted pipe.

Manufacturers generally guarantee that the efficiency of the weld will be 90 per cent, but many tests show that the weld is practically as strong as the plate. It has been found that the elastic limit of the steel is altered very little by welding.

215. Loading.—Methods of determining the loading, or maximum internal pressure, for closed conduits are given in Sec. 184. This loading influences particularly the thickness of plate and the longitudinal riveting. Special loadings, such as the internal pressure existing while the pipe is being filled, which tends to distort the pipe, the weight of water in the pipe, which must be carried by beam action to the piers, and the forces at anchorages, all of which affect the stress in saddles, piers, and circumferential riveting, are treated in Sec. 221.

216. Determination of Diameter.—For successful operation, the size of the pipe, for a given discharge, may vary between wide limits; but there is usually one size that will make for greatest economy of design. The reader is referred to Sec. 87 for a discussion of the general theory of economic design, and to Sec. 183 for its application to the special case of conduits. It has been found that velocities in excess of 20 ft. per second do not injure the pipe or its lining.

Usual velocities are from 8 to 12 ft. per second for heads up to about 200 ft. For very high-head plants the economic velocity will be much higher.

The shape of the load curve will have considerable influence in determining the maximum economic velocity. Thus, if the peak load is of very short duration, it is evident that it will be economical to use a relatively high velocity during this period.

Several high-head plants in the West are using velocities of 15 to 20 ft. per second in steel penstocks, and the tendency is to use even higher velocities.

Enger [1] has derived an equation for the economic diameter of steel pipe lines. This equation, which is based on Adam's Theorem, stated in Sec. 87, and on the methods and nomenclature used in this book, is as follows:

$$d = 9.08\left\{\frac{ebCQ^3}{a(R+i)tc^2(1+n)}\right\}^{1/6}, \quad . \quad . \quad . \quad . \quad . \quad (134)$$

where d = the most economical diameter, in inches;
- e = the over-all efficiency of the plant to point of sale, expressed as a decimal;
- b = value of lost energy, in mills per kilowatt-hour at point of sale;
- C = a coefficient for use in determining the productive head, as explained in Sec. 50;
- Q = the average discharge, in cubic feet per second;
- a = the cost of steel in the pipe, in dollars per pound;
- R = the desired per cent net return on money invested, expressed as a decimal;
- i = the estimated annual operating, tax, and depreciation charges, in per cent of construction cost, expressed as a decimal;
- t = the thickness of the pipe plate, in inches;
- c = Chezy's friction coefficient, which may be assumed, for riveted steel pipe, as 110;*
- n = per cent overweight due to laps, rivets, etc., expressed as a decimal.

To explain this equation by an example, let

$e = 0.75$	$a = 0.10$	$n = 0.2$
$b = 7.0$ mills	$R = 0.15$	
	$i = 0.02$	
$C = 2.0$	$t = 0.25$	
$Q = 200$ sec.-ft.	$c = 110$	

Then, for a thickness of 0.25 in.

$$d = 9.08\left\{\frac{0.75 \times 7 \times 2 \times 200 \times 200 \times 200}{0.1 \times 0.17 \times 0.25 \times 110 \times 110 \times 1.2}\right\}^{1/6},$$

$$d = 9.08(1{,}360{,}000)^{1/6},$$
$$d = 95.6 \text{ in.},$$

which would be called 96 in. for simplicity in design.

[1] Eng. Record, Vol. 70, p. 300.

* This value is close enough in consideration of other approximations in the equation explained hereinafter. For more exact information see "American Civil Engineers' Handbook," by Merriman. John Wiley & Sons, 4th Ed., p. 1089.

In Eq. (134), let A represent the constants for a given pipe line, or

$$A = \frac{ebCQ^3}{aRc^2(1 + n)}. \qquad (135)$$

Then Eq. (134) reduces to:

$$d = \left(\frac{A}{t}\right)^{1/6}. \qquad (136)$$

In order to apply Eq. (136) to a practical design, compute A, then determine the economical diameter for different values of t. With t and d known, the allowed head may be computed, and this determines the economical diameter at each station on the line where the head corresponds to the values of t and d. Thus, for the preceding example, the allowed head for a 96-in. pipe 0.25 in. thick is computed to be 130 ft. Therefore, at the station on the profile where the head is 130 ft., the diameter would be made 96 in.

The foregoing method for the direct determination of the economical diameter for steel pipe is not exact, because of certain approximations which necessarily had to be used in the derivation of Eq. (134). However, some of these approximations are necessary in the most exact determinations of economical diameter, and it is therefore considered that the method is close enough for ordinary practical purposes.

Equation (134) shows that the economical diameter varies inversely with the thickness. Therefore the most economical diameter is always smallest at the highest head.

217. Design of Steel Pipe.—Eq. (133) gives the tension per linear inch of pipe and the required thickness of steel plate to withstand the internal pressure. The efficiency of various types of riveted joints varies within a wide range, and the most economical type for given conditions depends upon the head and size of pipe. The most efficient joint is also the most expensive joint; and whether an increase in the cost of the joint to reduce the thickness of plate is justifiable, is a problem in relative economy.

Efficiencies of riveted steel joints are given in Figs. 282 to 287, inclusive.

The actual efficiency of welded steel pipe joints is between 90 and 100 per cent, and 90 per cent should be used in the calculations for strength.

Some engineers add about $\frac{1}{16}$ in. to the calculated thickness of plate to allow for possible corrosion; but this is questionable practice except for extremely thin plates, or perhaps for buried pipe. Exposed pipes are easily taken care of on the outside and experience has shown that pipes constantly full of water corrode very slowly on the inside even if the paint is allowed to deteriorate.

A minimum thickness of plate of $\frac{3}{8}$ in. is common practice, although a number of pipes have been installed with a minimum thickness of $\frac{1}{4}$ in. A minimum thickness is required to give sufficient stiffness at the saddles[3] and between them, and to provide for a reasonable amount of corrosion of the surface without too large a percentage reduction in thickness. Stiffeners are

[3] See Sec. 221.

sometimes provided, to increase the stability of the pipe, by riveting circumferential angles around the pipe as indicated in Fig. 292*A*.

Section 184 gives recommended working stresses for closed conduits in percentages of the elastic limit of the steel. The specified physical properties of steel recommended for steel pipe are given in Table XLV. The yield point given in this table is the "commercial elastic limit," the actual elastic limit being several thousand pounds greater. It is recommended, however, that the averages of the specified values of yield point, indicated in Table XLV, be used as the elastic limit in determining the working stresses in accordance with Sec. 184; and these averages, together with corresponding shear and bearing in rivets, are given in Table XLII.

TABLE XLII

RECOMMENDED VALUES FOR STRESSES AT THE ELASTIC LIMIT OF STEEL PIPE

Pounds per Square Inch

	Tension	Shop Rivets		Field Rivets	
		Shear	Bearing	Shear	Bearing
Welded pipe:					
Class A steel	24,000	24,000	48,000	20,000	40,000
Class B steel	27,000	24,000	48,000	20,000	40,000
Riveted Pipe	30,000*	24,000	48,000	20,000	40,000

* The plate mills will meet this figure for yield point.

Standard Details for Riveted Joints for Steel Pipe. Assumptions used in Design. Pacific Coast Electric Association.[3]

Materials.—Plate shall conform to standard specifications of American Society for Testing Materials for boiler flange steel.

Rivets shall conform to standard specifications of American Society for Testing Materials for boiler rivet steel.

Stresses

Type	Ultimate Strength[4] Lbs. per Square Inch
Tension	55,000
Shear	44,000
Bearing	95,000

Details

Hole diameter shall be $\frac{1}{16}$ in. larger than shank of cold rivet and is used in computing shearing and bending strength of rivets.

Punching and reaming: All holes for butt-joint pipe shall be sub-punched and reamed. All holes for lap-joint pipe of $\frac{7}{16}$ in. thickness or more shall be sub-punched and reamed. Holes in lap-joint pipe of $\frac{3}{8}$ in. thickness or less may be punched to size.

[3] From Report of Hydraulic Power Committee, National Electric Light Association, 1923, p. 32.

[4] EDITOR'S NOTE: It is recommended, as previously stated, that the factor of safety be based on the elastic limit instead of the ultimate strength.

Deductions for net area: For punched holes, a deduction for hole of $\frac{3}{16}$ in. greater diameter than cold rivet shank diameter is made in computing net area. For sub-punched and reamed holes, a deduction for hole of $\frac{1}{16}$ in. greater diameter than cold rivet shank diameter is made in computing net area.

Edge distances: Edge distances are at least 1.5 times diameter of hole.

Rivet spacing: The distances between rows of rivets is such that sum of the two net diagonal dimensions between holes will not be less than 1.25 times the net distance between holes on gage lines.

The maximum spacing of holes along calked edges is governed by the formula, $P = 2\frac{1}{2}t + d + 1\frac{1}{2}$ in., where t is plate thickness, d is diameter of rivet holes, p is pitch.

All rivet spacings shall be great enough to permit of use of standard rivet dies.

The standards for riveted joints shown in Figs. 282 to 287, inclusive, are those of the Pacific Coast Electrical Association, as published by the National Electric Light Association, 1913.

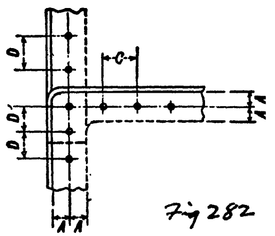


Plate Thickness	Diameter, Rivet	A	C	D	Efficiency, Plate	Efficiency, Rivet B	Efficiency, Rivet S
$\frac{1}{8}$	$\frac{3}{8}$	$\frac{11}{16}$	$1\frac{5}{16}$	$1\frac{5}{16}$	57.3	57.8	73.5
$\frac{3}{16}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{3}{4}$	$1\frac{3}{4}$	60.6	55.6	60.5
$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{1}{16}$	2	2	59.3	59.3	59.3
$\frac{5}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$	58.3	62.2	58.8
$\frac{3}{8}$	$\frac{7}{8}$	$1\frac{7}{16}$	$2\frac{5}{8}$	$2\frac{5}{8}$	60.0	61.1	55.5
$\frac{7}{16}$	$\frac{7}{8}$	$1\frac{7}{16}$	$2\frac{5}{8}$	$2\frac{5}{8}$	64.3	52.8	48.0
$\frac{1}{2}$	1	$1\frac{5}{8}$	3	3	64.5	61.1	47.2
$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{13}{16}$	$3\frac{3}{8}$	$3\frac{3}{8}$	64.8	60.8	46.7
$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{3}{4}$	$3\frac{3}{4}$	64.9	60.5	46.2

FIG. 282.—Standard Details for Riveted Joints for Steel Pipe Single-riveted Lap Joints. Pacific Coast Electrical Association.

Pipe-detail standards of a large engineering corporation are given in Fig. 292. This figure does not cover as wide a range as Figs. 282 to 287, inclusive, but the latter include no weight and strength data. An example showing the method of using Fig. 292 is given in Sec. 218.

The report of the Hydraulic Power Committee of the National Electric Light Association, 1923, recommends the use of bump joints for welded pipe. Flange joints are common for very small pipe. They permit of slight deviation in alinement. Double riveting of the joint is recommended. Rivet holes should be drilled $\frac{1}{16}$ in. small in the shop and reamed in the field after

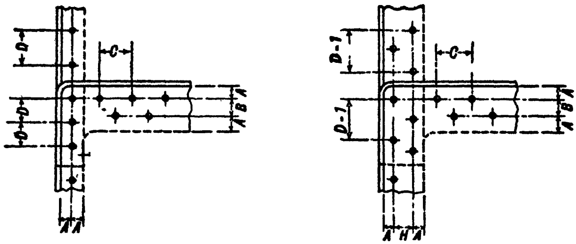


Plate Thickness	Diameter, Rivet	A	B	C	D	D-1	H	Efficiency, Plate	Efficiency, Rivet B	Efficiency, Rivet S
$\frac{1}{8}$	$\frac{3}{8}$	$\frac{11}{16}$	$1\frac{1}{16}$	$2\frac{1}{16}$	$1\frac{5}{16}$	.....	.....	72.8	73.3	93.2
$\frac{3}{16}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{1}{4}$	$2\frac{1}{2}$	$1\frac{3}{4}$	.....	.....	72.6	77.8	84.6
$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{1}{16}$	$1\frac{7}{16}$	$2\frac{13}{16}$	2	.....	.....	71.3	84.5	84.5
$\frac{5}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{11}{16}$	$3\frac{1}{16}$	$2\frac{1}{4}$	.....	.....	69.5	91.7	86.7
$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{8}$	$2\frac{1}{4}$	.....	.....	70.0	90.0	70.7
$\frac{7}{16}$	$\frac{7}{8}$	$1\frac{7}{16}$	2	$3\frac{7}{16}$	.....	$3\frac{7}{16}$	2	72.5	94.0	73.4
$\frac{1}{2}$	1	$1\frac{5}{8}$	$2\frac{3}{8}$	$3\frac{13}{16}$	.....	$3\frac{13}{16}$	$2\frac{3}{8}$	72.2	96.1	74.5
$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{13}{16}$	$2\frac{11}{16}$	$4\frac{1}{16}$	.....	$4\frac{1}{16}$	$2\frac{11}{16}$	70.7	101.0	77.4
$\frac{5}{8}$	$1\frac{1}{8}$	$1\frac{13}{16}$	$2\frac{11}{16}$	4	.....	$4\frac{3}{16}$	$2\frac{11}{16}$	70.3	106.6	70.7
$\frac{11}{16}$	$1\frac{1}{4}$	2	3	$4\frac{1}{2}$	.....	$4\frac{1}{2}$	3	70.8	100.6	69.7
$\frac{3}{4}$	$1\frac{1}{4}$	2	$3\frac{1}{8}$	$4\frac{3}{16}$	.....	$4\frac{5}{8}$	3	68.7	108.0	68.8

FIG. 283.—Standard Details for Riveted Joints for Steel Pipe Double-riveted Lap Joints. Pacific Coast Electrical Association.

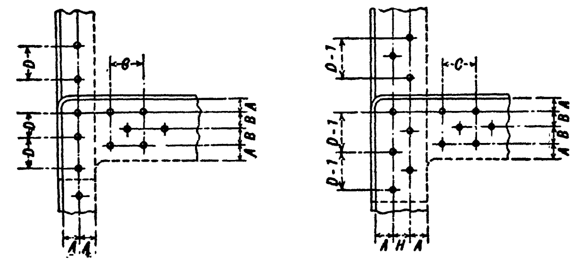
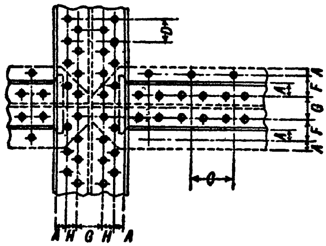


Plate Thickness	Diameter, Rivet	A	B	C	D	D-1	H	Efficiency, Plate	Efficiency, Rivet B	Efficiency, Rivet S
$\frac{1}{4}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{5}{16}$	$2\frac{11}{16}$	2	.....	.....	74.5	108.0	88.7
$\frac{5}{16}$	$\frac{5}{8}$	$1\frac{1}{16}$	$1\frac{5}{8}$	3	$2\frac{1}{4}$	.....	.....	73.0	118.6	95.0
$\frac{3}{8}$	$\frac{5}{8}$	$1\frac{1}{16}$	$1\frac{1}{2}$	$3\frac{1}{16}$	$2\frac{5}{8}$	.....	.....	73.5	116.4	77.5
$\frac{7}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{7}{16}$	.....	$3\frac{7}{16}$	2	76.2	125.0	82.8
$\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{5}{16}$	.....	$3\frac{1}{2}$	2	76.0	127.0	75.2
$\frac{9}{16}$	$\frac{7}{8}$	$1\frac{7}{16}$	2	$3\frac{13}{16}$	.....	$3\frac{13}{16}$	2	75.5	127.4	77.2
$\frac{5}{8}$	$\frac{7}{8}$	$1\frac{7}{16}$	$1\frac{15}{16}$	$3\frac{9}{16}$	.....	4	$2\frac{1}{16}$	73.7	133.0	74.3
$\frac{11}{16}$	1	$1\frac{5}{8}$	$2\frac{3}{16}$	$4\frac{1}{8}$	.....	$4\frac{1}{4}$	$2\frac{1}{4}$	74.2	133.0	75.0
$\frac{3}{4}$	$1\frac{1}{8}$	$1\frac{13}{16}$	$2\frac{1}{2}$	$4\frac{9}{16}$	.....	$4\frac{9}{16}$	$2\frac{1}{2}$	73.9	134.7	77.5
$\frac{13}{16}$	$1\frac{1}{8}$	$1\frac{13}{16}$	$2\frac{1}{2}$	$4\frac{1}{2}$	.....	$4\frac{11}{16}$	$2\frac{7}{16}$	73.3	136.5	72.4
$\frac{7}{8}$	$1\frac{1}{4}$	2	$2\frac{13}{16}$	5	.....	5	$2\frac{13}{16}$	73.6	136.0	74.2
$\frac{15}{16}$	$1\frac{1}{4}$	2	$2\frac{7}{8}$	$4\frac{3}{4}$	.....	$5\frac{1}{8}$	$2\frac{3}{4}$	72.4	143.0	73.0
1	$1\frac{1}{4}$	2	3	$4\frac{9}{16}$	.....	$5\frac{1}{4}$	$2\frac{3}{4}$	71.3	149.0	71.3

FIG. 284.—Standard Details for Riveted Joints for Steel Pipe Triple-riveted Lap Joints. Pacific Coast Electrical Association.

the pipe is in place. Pipe smaller than 26 in. in diameter should always be flanged. Rivet pass holes should be left in every second length of pipe, and they should be fitted with screw plugs of brass or malleable iron.

A few types of joints for welded pipe are indicated in Fig. 288. Other types for special conditions are fabricated.



Thickness				A	B	C	D	F	G	H	Efficiency		
Pipe Plate	Long. Strap	Cir. Strap	Dia. Riv.								Out. Row	Inside Row	Rivet
$\frac{3}{8}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{15}{16}$	$4\frac{3}{4}$	$3\frac{1}{8}$	$2\frac{11}{16}$	$2\frac{3}{4}$	$1\frac{5}{8}$	83.0	89.0	82.4
$\frac{7}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{7}{8}$	$1\frac{7}{16}$	$2\frac{1}{16}$	$5\frac{7}{16}$	$3\frac{3}{8}$	3	$3\frac{1}{8}$	$1\frac{7}{8}$	82.8	86.9	82.8
$\frac{1}{2}$	$\frac{7}{16}$	$\frac{7}{16}$	1	$1\frac{9}{16}$	$2\frac{9}{16}$	$6\frac{1}{8}$	$3\frac{5}{8}$	$3\frac{5}{16}$	$3\frac{3}{8}$	$2\frac{7}{16}$	82.6	88.5	83.1
$\frac{9}{16}$	$\frac{1}{2}$	$\frac{1}{2}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$2\frac{15}{16}$	$6\frac{7}{8}$	$3\frac{15}{16}$	$3\frac{11}{16}$	$3\frac{3}{4}$	$2\frac{3}{4}$	82.8	88.4	82.5
$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$1\frac{1}{4}$	2	$3\frac{1}{4}$	$7\frac{9}{16}$	$4\frac{1}{16}$	$4\frac{1}{16}$	$4\frac{1}{4}$	$3\frac{3}{16}$	82.6	88.0	82.7
$\frac{11}{16}$	$\frac{9}{16}$	$\frac{9}{16}$	$1\frac{1}{4}$	2	$3\frac{1}{4}$	$7\frac{7}{16}$	$4\frac{1}{4}$	$4\frac{1}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	82.2	85.8	82.1
$\frac{3}{4}$	$\frac{9}{16}$	$\frac{9}{16}$	$1\frac{1}{4}$	2	$3\frac{5}{16}$	$7\frac{1}{4}$	$4\frac{1}{4}$	$4\frac{1}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	81.9	83.5	82.5
$\frac{13}{16}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{5}{16}$	$7\frac{3}{16}$	$4\frac{3}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	$3\frac{1}{16}$	81.6	82.0	81.6
$\frac{7}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{5}{16}$	$7\frac{1}{8}$	$4\frac{3}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	$3\frac{1}{16}$	81.6	80.5	80.8
$\frac{15}{16}$	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$3\frac{5}{16}$	$7\frac{1}{8}$	$4\frac{5}{8}$	$4\frac{3}{16}$	$4\frac{1}{4}$	3	81.5	79.0	79.7
1	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$3\frac{5}{16}$	7	$4\frac{5}{8}$	$4\frac{3}{16}$	$4\frac{1}{4}$	3	81.1	78.0	77.4

FIG. 285.—Standard Details for Riveted Joints for Steel Pipe Double-riveted Double Butt Joints.

Pacific Coast Electrical Association.

Flanges of penstocks should be ordered and drilled to fit those of the valves or wheel casings to which they are to be fastened. The design should permit of having all bolts accessible. Bolt holes should be spot-faced.

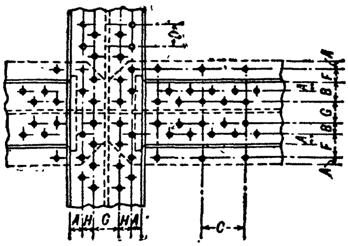
For practically all pressures, a round gutta percha gasket about $\frac{1}{4}$ in. in diameter, with ends scarfed and cemented, is recommended by the Hydraulic Power Committee of the National Electric Light Association as being the most satisfactory. Such a gasket is placed in a groove in the face of one of the companion flanges, as indicated in Fig. 290 and cemented in place with rubber cement. Only the face of one flange is grooved and the other is left flat.

218. Estimating Weights of Steel Pipe.—Figure 291, computed for this book by Mr. Gardner C. George, shows the strength, weight, and most economical riveting of steel pipe, calculated according to certain assumptions which will be described later. From the intersection of the stepped lines representing diameter of pipe with the inclined lines representing thickness and riveting,

trace vertically to the margin, and read the head corresponding to 15,000 lb. per square inch stress; or trace horizontally to the margin, and read weight of pipe per linear foot. If the thickness and riveting are not desired, trace vertically from the given head to intersect the given diameter, thence horizontally to the margin, and read weight of pipe.

As an example, to show the application of Fig. 291, assume:

Maximum head, 210 ft.
Diameter, 10 ft.
Allowed stress, 15,000 lb. per square inch.



Thickness				A	B	C	D	F	G	H	Efficiency		
Pipe Plate	Long. Strap	Cir. Strap	Dia. Riv.								Out. Row	Inside Row	Rivet
$\frac{3}{8}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{8}$	1	$1\frac{1}{2}$	6	3	$2\frac{3}{8}$	$2\frac{1}{4}$	$1\frac{3}{8}$	88.5	90.4	92.5
$\frac{7}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{5}{8}$	1	$1\frac{1}{2}$	$6\frac{1}{8}$	$2\frac{3}{4}$	$2\frac{3}{8}$	$2\frac{1}{4}$	$1\frac{1}{2}$	88.8	88.6	88.5
$\frac{1}{2}$	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$6\frac{7}{8}$	$3\frac{1}{8}$	$2\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{5}{8}$	88.4	88.5	93.3
$\frac{9}{16}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{7}{16}$	$1\frac{7}{8}$	$7\frac{3}{8}$	$3\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{1}{8}$	$2\frac{1}{16}$	87.2	87.8	100.0
$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{7}{16}$	$1\frac{7}{8}$	$7\frac{3}{8}$	$3\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{1}{8}$	$2\frac{1}{16}$	87.4	86.4	99.4
$\frac{11}{16}$	$\frac{9}{16}$	$\frac{9}{16}$	1	$1\frac{9}{16}$	$2\frac{1}{4}$	8	$3\frac{5}{8}$	$3\frac{3}{8}$	$3\frac{3}{8}$	$2\frac{7}{16}$	86.9	86.5	100.0
$\frac{3}{4}$	$\frac{9}{16}$	$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$2\frac{3}{8}$	$8\frac{1}{4}$	$4\frac{1}{8}$	$3\frac{3}{4}$	$3\frac{3}{4}$	$2\frac{11}{16}$	85.6	85.5	100.0
$\frac{13}{16}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{1}{16}$	$8\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	$3\frac{1}{16}$	85.0	85.4	100.0
$\frac{7}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{1}{16}$	$8\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	$3\frac{1}{16}$	85.0	84.0	100.0
$\frac{15}{16}$	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$2\frac{15}{16}$	$9\frac{3}{8}$	$4\frac{1}{4}$	$4\frac{3}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	86.0	84.5	100.0
1	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$2\frac{15}{16}$	$9\frac{3}{8}$	$4\frac{1}{4}$	$4\frac{3}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	86.3	83.6	100.0
$1\frac{1}{16}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$	2	$2\frac{13}{16}$	10	$3\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{1}{4}$	$3\frac{1}{4}$	86.9	84.0	91.8
$1\frac{1}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$	2	$2\frac{13}{16}$	10	$3\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{1}{4}$	$3\frac{1}{4}$	86.7	83.5	86.6
$1\frac{3}{16}$	1	1	$1\frac{1}{4}$	2	$2\frac{13}{16}$	10	$3\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{4}$	$3\frac{1}{4}$	86.7	82.8	82.2
$1\frac{1}{4}$	1	1	$1\frac{1}{4}$	2	$2\frac{7}{8}$	$9\frac{1}{2}$	$3\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{4}$	$3\frac{1}{4}$	86.4	81.6	82.1

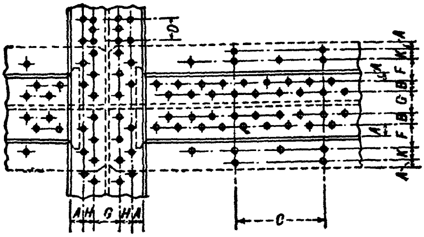
Fig. 286.—Standard Details for Riveted Joints for Steel Pipe Triple-riveted Double Butt Joints.
Pacific Coast Electrical Association.

Locate 210 ft. head at the lower margin of the diagram and trace vertically to intersect the stepped line representing 10 ft. diameter, then trace horizontally to read weight of pipe 708 lb. per linear foot. The intersection of the 210 ft. head line and the 10 ft. diameter line lies between the two lines representing $\frac{7}{16}$-in. triple-riveted lap pipe and $\frac{1}{2}$-in. triple-riveted lap pipe. The former would be too weak, as the diagram shows it to be good for only 196

ft. head, while the latter is good for 215 ft. head. Hence the weight given is computed for the stronger pipe.

The diagram may be used for other stresses than 15,000 lb., as follows. In the preceding example, suppose a stress of 20,000 lb. is allowed. Then enter the diagram with a head computed as follows, and proceed as before described.

$$\text{Head to use in diagram} = 210 \times \frac{15{,}000}{20{,}000} = 158 \text{ ft. head.}$$



Thickness				A	B	C	D	F	G	H	K	Efficiency			
Pipe	Long. Strap	Cir. Strap	Dia. Riv.									1st Row	2nd Row	3rd Row	Rivet
$\frac{3}{8}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{8}$	1	$1\frac{1}{2}$	12	3	$2\frac{3}{8}$	$2\frac{1}{4}$	$1\frac{3}{8}$	2	94.0	93.6	94.4	100.0
$\frac{7}{16}$	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{11}{16}$	13	$3\frac{1}{8}$	$2\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{5}{8}$	$2\frac{1}{4}$	93.6	94.6	96.7	100.0
$\frac{1}{2}$	$\frac{7}{16}$	$\frac{7}{16}$	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	$13\frac{1}{2}$	$3\frac{1}{8}$	$2\frac{3}{4}$	$2\frac{3}{4}$	$1\frac{5}{8}$	$2\frac{1}{4}$	93.8	94.0	94.2	100.0
$\frac{9}{16}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{7}{16}$	$1\frac{7}{8}$	$14\frac{3}{4}$	$3\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{1}{8}$	$2\frac{1}{16}$	$2\frac{5}{8}$	93.6	93.8	94.5	100.0
$\frac{5}{8}$	$\frac{1}{2}$	$\frac{1}{2}$	$\frac{7}{8}$	$1\frac{7}{16}$	$1\frac{7}{8}$	$14\frac{3}{4}$	$3\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{1}{8}$	$2\frac{1}{16}$	$2\frac{5}{8}$	93.6	93.0	92.5	100.0
$\frac{11}{16}$	$\frac{9}{16}$	$\frac{9}{16}$	1	$1\frac{9}{16}$	$2\frac{1}{4}$	16	$3\frac{5}{8}$	$3\frac{3}{8}$	$3\frac{3}{8}$	$2\frac{7}{16}$	3	93.4	93.1	92.7	100.0
$\frac{3}{4}$	$\frac{9}{16}$	$\frac{9}{16}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$2\frac{3}{4}$	$16\frac{1}{4}$	$4\frac{1}{8}$	$3\frac{3}{4}$	$3\frac{3}{4}$	$2\frac{11}{16}$	$3\frac{3}{8}$	92.6	92.5	92.5	100.0
$\frac{13}{16}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{8}$	$1\frac{3}{4}$	$2\frac{5}{8}$	17	$4\frac{1}{4}$	$3\frac{13}{16}$	$3\frac{3}{4}$	$2\frac{5}{8}$	$3\frac{3}{8}$	93.2	92.5	91.3	100.0
$\frac{7}{8}$	$\frac{5}{8}$	$\frac{5}{8}$	$1\frac{1}{4}$	2	$3\frac{1}{16}$	$17\frac{1}{2}$	$4\frac{3}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	$3\frac{1}{16}$	$3\frac{3}{4}$	92.4	92.0	91.2	100.0
$\frac{15}{16}$	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$2\frac{15}{16}$	$18\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{3}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{3}{4}$	93.1	92.2	90.4	100.0
1	$\frac{3}{4}$	$\frac{3}{4}$	$1\frac{1}{4}$	2	$2\frac{15}{16}$	$18\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{3}{16}$	$4\frac{1}{4}$	$3\frac{1}{8}$	$3\frac{3}{4}$	93.1	91.7	89.4	100.0
$1\frac{1}{16}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$	2	$2\frac{13}{16}$	20	$3\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{1}{4}$	$3\frac{1}{4}$	$3\frac{3}{4}$	92.6	91.9	89.0	97.0
$1\frac{1}{8}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{4}$	2	$2\frac{13}{16}$	20	$3\frac{3}{4}$	$4\frac{1}{4}$	$4\frac{1}{4}$	$3\frac{1}{4}$	$3\frac{3}{4}$	93.1	91.5	88.0	91.3
$1\frac{3}{16}$	1	1	$1\frac{1}{4}$	2	$2\frac{13}{16}$	20	$3\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{4}$	$3\frac{1}{4}$	$3\frac{3}{4}$	93.3	91.3	87.4	86.6
$1\frac{1}{4}$	1	1	$1\frac{1}{4}$	2	$2\frac{7}{8}$	$19\frac{1}{4}$	$3\frac{3}{4}$	$4\frac{3}{8}$	$4\frac{1}{4}$	$3\frac{1}{4}$	$3\frac{3}{4}$	93.0	91.9	86.0	85.4

FIG. 287.—Standard Details for Riveted Joints for Steel Pipe Quadruple-riveted Double Butt Joints.
Pacific Coast Electrical Association.

The diagram of Fig. 291 was computed from the data contained in Fig. 292, which are the standards of a large engineering corporation. The use of Fig. 292 and the method of computing Fig. 291 will now be explained by means of an example, the same basic data being used as in the example explaining Fig. 291, namely,

Maximum head,	210 ft.
Diameter,	10 ft.
Allowed stress,	15,000 lb. per square inch.

From Eq. (132), the tension per linear inch is

$$T = 2.6 \times 210 \times 10 = 5460 \text{ lb.},$$

and the tension per linear foot is

$$T' = 12 \times 5460 = 65{,}520 \text{ lb.}^{5}$$

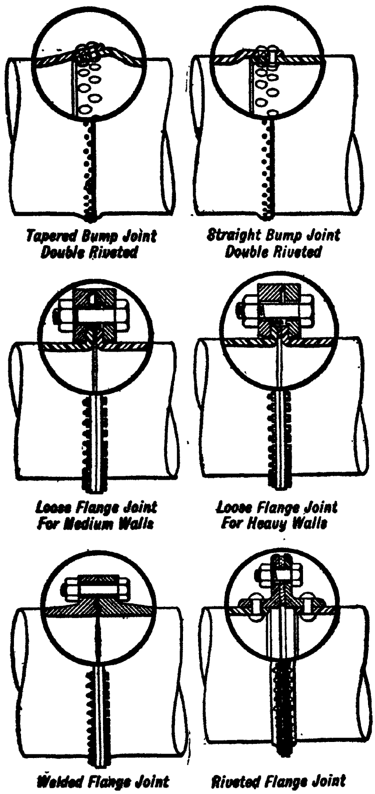


FIG. 288.—Typical Joints for Welded Pipe National Tube Company.

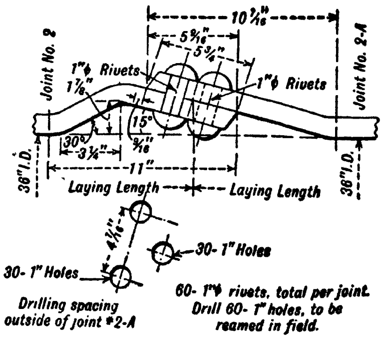


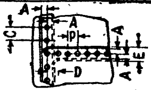
FIG. 289.—Typical Double-riveted Bump Joint.
1923 Report, Hydraulic Power Committee. National Electrical Light Association.

FIG. 290.—Showing Grooved Face of Flange for Use with Round Gutta Percha Gasket.

Pipe characteristics for different types of joints are given in Table XLIII. The thickness (Col. 2) and the excess weight of one longitudinal joint (Col. 4) are obtained from Fig. 292 opposite the next highest tension to 65,520 lb. The plate weight (Col. 3) is obtained from the table of plate weights of Fig. 292 by multiplying the weight, for the given thickness, by the diameter of the pipe.

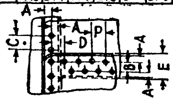
[5] Fig. 292 is based on 15,000 lb. per square inch tension. If, for instance, 20,000 lb. per square inch is the allowed tension, multiply T' by $\frac{15{,}000}{20{,}000}$ before using the figure.

Thickness of Plate	Rivet Dia	Hole Dia	P	A	C	D	E	Joint Eff.	Tension per Lin. Ft of Pipe	Excess Wt per Ft of Ilg Joint	Wt per Ft of Girt Joint
3/16	5/8	11/16	1 7/8	1 1/16	1 7/8±	2	2 3/8	58.6	19,750	2.60#	2.60
1/4	5/8	11/16	1 7/8	1 1/16	1 7/8±	2	2 3/8	55.5	25,000	3.11	3.11
5/16	3/4	13/16	2 1/4	1 1/4	2 1/4±	2 1/4	2 3/4	51.5	29,000	4.44	4.44



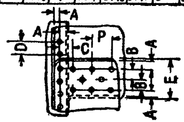
Single Riveted Lap

Thickness Pipe Plate	Rivet Dia	Hole Dia.	P	A	B	C	D	E	Joint Eff	Tension per Lin. Ft of Pipe	Excess Wt per Ft of Ilg Joint	Wt per Ft of Girt Joint
3/16	1/2	9/16	2 7/16	7/8	1 1/4	1 3/4±	2	3 1/4	73.9	24,950	2.93*	1.91
1/4	5/8	11/16	2 3/4	1 1/16	1 1/2	1 7/8±	2	3 7/8	75.0	33,750	4.78	3.11
5/16	3/4	13/16	3 1/8	1 1/4	1 5/8	2 1/4±	2 1/2	4 3/8	74.0	41,600	6.84	4.44
3/8	3/4	13/16	2 3/4	1 1/4	1 5/8	2 1/4±	2 1/2	4 3/8	70.3	47,500	8.07	5.07
7/16	7/8	15/16	3 1/8	1 7/16	1 7/8	2 5/8±	3	5	70.0	55,000	10.71	6.61
1/2	1	1 1/16	3 1/2	1 3/4	2 1/8	3±	3 1/4	5 7/8	69.7	62,700	14.25	8.86
9/16	1	1 1/16	3 1/4	1 3/4	2 1/8	3±	3 1/4	5 7/8	67.4	68,300	15.84	9.63



Double Riveted Lap

Thickness Pipe Plate	Rivet Dia	Hole Dia	P	A	B	C	D	E	Joint Eff.	Tension per Lin. Ft of Pipe	Excess Wt per Ft. of Ilg Joint	Wt per Ft of Girt. Joint
5/16	3/4	13/16	4 1/4	1 1/4	2 1/8	2 1/2±	2 1/4±	7	80.9	45,500	9.85	4.44
3/8	3/4	13/16	3 3/4	1 1/4	2 1/8	2 1/2±	2 1/4±	7	77.5	52,300	11.67	5.07
7/16	7/8	15/16	4 1/4	1 7/16	2 3/8	3±	2 5/8±	7 7/8	77.9	61,300	15.32	6.61
1/2	7/8	15/16	3 7/8	1 7/16	2 3/8	3±	2 5/8±	7 7/8	75.0	67,500	17.34	7.27
9/16	7/8	15/16	3 1/2	1 7/16	2 3/8	3±	2 5/8±	7 7/8	73.2	74,200	19.43	7.88
5/8	1	1 1/16	4	1 3/4	2 11/16	3 1/4±	3±	9 1/8	73.5	82,700	24.97	10.42



Triple Riveted Lap

Stress in # □" Used in Design

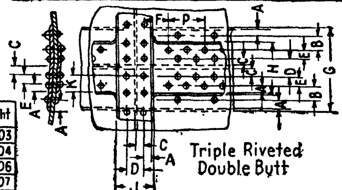
Tension	*15,000*
Single Shear	*10,500*
Double Shear	*20,000*
Bearing	*24,000*

Table of Weights Plates 12" Wide 3.1416 Lg

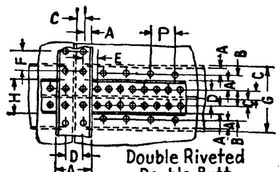
Thick	Weight
3/16	24.03
1/4	32.04
5/16	40.06
3/8	48.07
7/16	56.08
1/2	64.09
9/16	72.10
5/8	80.11
11/16	88.12
3/4	96.13
13/16	104.14
7/8	112.16
15/16	120.17
1	128.18
1 1/16	136.19
1 1/8	144.20
1 3/16	152.21
1 1/4	160.22

Triple Riveted Double Butt

Thickness Pipe Plate	Thickness Long Strap	Thickness Gir Strap	Rivet Dia	Hole Dia	P	A	B	C	D	E	F	G	H	J	K	Joint Eff	Tension per Lin Ft of Pipe	Excess Wt per Ft of Ilg Joint	Wt per Ft of Girt Joint
3/8	5/16	7/16	3/4	13/16	6 7/8	1 1/4	2 1/2	1 5/16	2 7/8	1 3/8	2 1/2±	13 7/8	8 7/8	5 5/8	2 1/4±	88.1	59,500	27.2#	11.6
7/16	3/8	1/2	7/8	15/16	7 7/8	1 7/16	2 7/8	1 1/2	3 1/4	1 7/8	3±	15 3/8	10 3/8	6 3/8	2 3/4±	88.1	69,400	37.0	15.1
1/2	7/16	9/16	7/8	15/16	7 7/8	1 7/16	2 7/8	1 1/2	3 1/4	1 7/8	3±	15 3/8	10 3/8	6 3/8	2 3/4±	88.1	79,300	42.6	16.3
9/16	1/2	5/8	1	1 1/16	8 7/8	1 3/4	3 1/2	1 11/16	3 7/8	2 1/8	3 1/4±	18 7/8	11 7/8	7 5/8	3±	88.0	89,000	57.3	21.5
5/8	1/2	11/16	1	1 1/16	8 7/8	1 3/4	3 1/2	1 11/16	3 7/8	2 1/8	3 1/4±	18 7/8	11 7/8	7 5/8	3±	87.2	98,200	57.3	23.1
11/16	9/16	3/4	1	1 1/16	8 3/4	1 3/4	3 1/2	1 11/16	3 7/8	2 1/8	3 1/4±	18 7/8	11 7/8	7 5/8	3±	86.0	106,500	63.9	24.8
3/4	9/16	13/16	1	1 1/16	8 1/8	1 3/4	3 1/2	1 11/16	3 7/8	2 1/8	3 1/4±	18 7/8	11 7/8	7 5/8	3±	84.0	113,400	64.3	26.4
13/16	5/8	7/8	1 1/8	1 3/16	9 3/8	1 15/16	3 7/8	2	4 1/4	2 3/8	3 1/2±	20 7/8	13 3/8	8 5/8	3 1/4±	84.8	124,000	78.9	32.1
7/8	5/8	15/16	1 1/8	1 3/16	8 3/4	1 15/16	3 7/8	2	4 1/4	2 3/8	3 1/2±	20 7/8	13 3/8	8 5/8	3 1/4±	82.9	130,800	79.4	33.8
15/16	3/4	1	1 1/8	1 3/16	8 1/4	1 15/16	3 7/8	2	4 1/4	2 3/8	3 1/2±	20 7/8	13 3/8	8 5/8	3 1/4±	81.2	137,000	94.3	35.6
1	3/4	1 1/16	1 1/8	1 3/16	7 7/8	1 15/16	3 7/8	2	4 1/4	2 3/8	3 1/2±	20 7/8	13 3/8	8 5/8	3 1/4±	79.7	143,500	94.6	37.2
1 1/16	7/8	1 1/8	1 1/4	1 5/16	9	2 1/8	4 1/4	2 3/16	4 5/8	2 1/2	3 3/4±	22 5/8	14 1/8	9 5/8	3 1/2±	80.7	154,500	119.6	44.5
1 1/8	7/8	1 3/16	1 1/4	1 5/16	8 5/8	2 1/8	4 1/4	2 3/16	4 5/8	2 1/2	3 3/4±	22 5/8	14 1/8	9 5/8	3 1/2±	79.3	161,000	120.1	46.3
1 3/16	1	1 1/4	1 1/4	1 5/16	8 1/4	2 1/8	4 1/4	2 3/16	4 5/8	2 1/2	3 3/4±	22 5/8	14 1/8	9 5/8	3 1/2±	77.9	166,900	136.2	48.2
1 1/4	1	1 5/16	1 1/4	1 5/16	7 7/8	2 1/8	4 1/4	2 3/16	4 5/8	2 1/2	3 3/4±	22 5/8	14 1/8	9 5/8	3 1/2±	76.3	171,800	136.8	50.1



Triple Riveted Double Butt



Double Riveted Double Butt

Thickness Pipe Plate	Thickness Long Strap	Thickness Gir. Strap	Rivet Dia.	Hole Dia.	P	A	B	C	D	E	F	G	H	J	Joint Eff.	Tension per Lin. Ft. of Pipe	Excess Wt. per Ft of Ilg Joint	Wt. per Ft. of Girt. Joint
3/8	5/16	7/16	3/4	13/16	4 3/8	1 1/4	2 1/2	1 5/16	2 7/8	2 1/2	2 1/4±	10 5/8	5 5/8	5 5/8	81.5	55,000	20.4#	11.6
7/16	3/8	1/2	7/8	15/16	5	1 7/16	2 7/8	1 1/2	3 1/4	3	2 5/8±	12 1/8	6 3/8	6 3/8	81.3	64,000	27.7	15.1
1/2	7/16	9/16	7/8	15/16	5	1 7/16	2 7/8	1 1/2	3 1/4	3	2 5/8±	12 1/8	6 3/8	6 3/8	79.3	71,400	31.6	16.3
9/16	1/2	5/8	1	1 1/16	5 1/2	1 3/4	3 1/2	1 11/16	3 7/8	3 1/4	3±	14 5/8	7 5/8	7 5/8	80.7	81,500	43.2	21.5
5/8	1/2	11/16	1	1 1/16	5 1/2	1 3/4	3 1/2	1 11/16	3 7/8	3 1/4	3±	14 5/8	7 5/8	7 5/8	79.4	89,400	43.2	23.1
11/16	9/16	3/4	1	1 1/16	5 1/2	1 3/4	3 1/2	1 11/16	3 7/8	3 1/4	3±	14 5/8	7 5/8	7 5/8	77.8	96,400	48.0	24.8
3/4	9/16	13/16	1	1 1/16	5 1/2	1 3/4	3 1/2	1 11/16	3 7/8	3 1/4	3±	14 5/8	7 5/8	7 5/8	72.2	97,600	48.0	26.4
13/16	5/8	7/8	1 1/8	1 3/16	6 1/8	1 15/16	3 7/8	2	4 1/4	3 1/2	3 1/4±	16 1/8	8 3/8	8 3/8	74.9	109,500	58.8	32.1
7/8	5/8	15/16	1 1/8	1 3/16	5 7/8	1 15/16	3 7/8	2	4 1/4	3 1/2	3 1/4±	16 1/8	8 3/8	8 3/8	72.5	114,300	59.1	33.8
15/16	3/4	1	1 1/4	1 5/16	6 1/2	2 1/8	4 1/4	2 3/16	4 5/8	3 3/4	3 1/2±	17 5/8	9 1/8	9 1/8	74.7	126,000	77.7	40.7
1	3/4	1 1/16	1 1/4	1 5/16	6 1/4	2 1/8	4 1/4	2 3/16	4 5/8	3 3/4	3 1/2±	17 5/8	9 1/8	9 1/8	73.0	131,500	78.1	42.6

FIG. 292.—Standard Joints for Riveted Plate Steel Pipes.

The number of longitudinal joints (Col. 5) corresponds to the width of plate used in the fabrication of the pipe. Usually the number of joints in the circumference is as follows:

One joint to 5.5 ft. diameter,
Two joints from 5.5 ft. to 11.0 ft. diameter,
Three joints from 11.0 ft. to 16.0 ft. diameter,
Four joints from 16.0 ft. to 21.0 ft. diameter.

TABLE XLIII

TABULATION OF PIPE CHARACTERISTICS FOR EXPLANATORY EXAMPLE

1 Riveting	2 Thickness	3 Plate Weight	4 Excess Weight One Long. Joint	5 No. of Long. Joints	6 Total Excess Weight of Long. Joint	7 Excess Weight Girth Joint per Lin. Ft. of Joint	8 Excess Weight, Girth Joint per Lin. Ft. of Pipe	9 Total Weight of Pipe. Sum of Cols. 3, 6, and 8
S. R. L.	Not applicable							
D. R. L.	$\frac{9}{16}$	721.0	15.84	2	31.7	9.63	43.2	795.9
T. R. L.	$\frac{1}{2}$	640.9	17.34	2	34.7	7.27	32.6	708.2
D. R. D. B.	$\frac{1}{2}$	640.9	31.60	2	63.2	16.3	73.2	777.2
T. R. D. B.	$\frac{7}{16}$	560.8	37.00	2	74.0	15.1	67.8	702.6

TABLE XLIV

TABULATION OF RELATIVE COSTS FOR EXPLANATORY EXAMPLE

1	2	3	4	5	6	7	8	9
		Weights			Relative Costs			
Riveting	Thickness Inch	Plate	Long. Joints	Girth Joints	Plate, Col. 3	Long Joints 2 Times Col. 4	Girth Joints 2.2 Times Col. 5	Total Relative Cost
D. R. L.	$\frac{9}{16}$	721.0	31.7	43.2	721.0	63.4	95.0	879.4
T. R. L.	$\frac{1}{2}$	640.9	34.7	32.6	640.9	69.4	71.7	782.0
D. R. D. B.	$\frac{1}{2}$	640.9	63.2	73.2	640.9	126.4	161.0	928.3
T. R. D. B.	$\frac{7}{16}$	560.8	74.0	67.8	560.8	148.0	149.2	858.0

The total excess weight of longitudinal joints (Col. 6) is the product of Cols. 4 and 5.

The excess weight of girth joints per linear foot (Col. 7) is taken from Fig. 292 opposite the thickness of pipe plate. The excess weight of girth joints per linear foot of pipe (Col. 8) is obtained from Col. 7 times the circumference of the pipe divided by the distance center to center of girth joints, which is usually about 7 or 8 ft. Seven feet is used here.

The total weight of pipe per linear foot (Col. 9) is the sum of the part weights given in Cols. 3, 6 and 8. The lowest resultant weight in Col. 9 does not necessarily indicate the most economical type of joint, because the thinner plates require a greater amount of shop work on account of their more intricate joints.

In order to determine the most economical joint, Table XLIV, showing the relative costs of pipe for various types of joints, has been prepared. Cols. 3, 4 and 5 are taken directly from Cols. 3, 6, and 8 of Table XLIII. Cols. 6, 7, and 8 are obtained by multiplying the weights of Cols. 3, 4 and 5 by the relative cost. In this example, the metal in the longitudinal joints is assumed to cost, in cents per pound, twice as much as the plate; and that in the girth joints, 2.2 times as much as the plate. The true cost of this metal depends, of course, upon various factors, chiefly upon whether the joint is fabricated in the shop or in the field, and upon the relative costs of steel and labor. It is very difficult to determine these factors, and shop experience is required to fix them closely.

Column 9 is the sum of Cols. 6, 7 and 8 and is an indication of the relative cost. The cost for the triple-riveted lap joint being the lowest, a ½-in. plate is adopted, and the total weight of pipe per linear foot, 708.2 lb., taken from Table XLIII.

With the foregoing assumptions, sufficient data were computed to plot Fig. 291.

219. Fabrication of Steel Pipe.—Plates used for steel pipe should be made of open-hearth steel. Table XLV gives the specifications of the American Society for Testing Materials, which are usually made to apply to the steel for pipes.

The following specifications for the fabrication of steel pipe are taken from the Report of the Hydraulic Power Committee of the National Electric Light Association, 1923.

SPECIFICATIONS FOR THE FABRICATION OF RIVETED STEEL PIPE

(*A*) General Specifications:

1. *Plates:*
   (*a*) All plates shall be free from laminations or surface defects and be up to gage on the edges, standard variations being allowed.
   (*b*) Any plate that develops defects during the process of punching, bending, and riveting incident to fabrication and erection of the pipe shall be rejected notwithstanding that the same may previously have satisfactorily passed specified test.
2. *Length of Sections:*
   In general riveted pipe shall be made in three-course sections, the total length of each section to be from 20 to 24 ft.
3. *Joints:*
   Details of all joints shall be in accordance with the standards shown on the accompanying drawings. All roundabout lap joints shall be constructed with female end uphill.
   Longitudinal lap joints shall point down and shall be located

TABLE XLV

MATERIALS FOR STEEL PIPES

Item	Serial Designation of Am. Soc. for Testing Materials	Specifications	Grade	Tensile Properties		
				Yield Point Minimum		
				Tensile Strength	Per cent of Tensile Strength	Not Less Than
Welded Pipe.....	*A*-78-23*T*	Tentative Specifications for Steel Plates of Structural Quality for Forged Welding.	*A* (Preferred) *B*	45,000 50,000	50 50	24,000 27,000
Riveted Steel Pipe	*A*-30-24	Standard Specifications for Boiler and Firebox Steel for Locomotives.	Flanged Steel Grade	55,000-65,000	50	
Rivets..........	*A*-31-24	Standard Specifications for Boiler-rivet Steel.	....	45,000-55,000	50	
Steel Castings....	*A*-27-24	Standard Specifications for Steel Castings.	Class B Medium Class B Soft (Preferred)	70,000 60,000	45 45	

alternately 30 degrees to the left and to the right of the top center line of the pipe.

Longitudinal butt joints shall be located at the top center of the pipe except where angle sections, air valves, or manholes occur, in which cases they shall be located near the horizontal diameter.

Butt straps for the roundabout joints shall be riveted to the pipe in the shop for one-half of the circumference less two rivets. The remaining portion is to be sub-punched but not reamed, and securely bolted for shipment.

All joints shall form a tight fit with each other. All angular joints shall be shop closed.

4. *Angle Sections:*

(*a*) Where angles or curves occur in either the alignment or the grade of the conduit, the plates must be cut and punched to the required lines for forming a small oblique angle at the roundabout seams, embracing as many courses as may be required to procure the total deflection or curvature, the courses being put together with the longitudinal seams staggered.

(*b*) In general the deflection angle formed by two consecutive courses may range from one (1) to five (5) degrees in the plane of the bend, according to locality, but greater deflection angles shall not be made except as specifically authorized by the Company. In general, no angle section shall consist of less than three courses.

(*c*) The work must be so laid out as to bring all longitudinal seams as near the horizontal diameter of the pipe as possible, preferably in the upper half of the section.

5. *Taper Courses:*

In forming taper courses, the plates must be cut and punched to the required lines along the four edges, so as to bring the pitch lines of the rivets in the roundabout seams into planes parallel with each other and at right angles to the axis of the section.

6. *Rivet Pass Holes:*

A rivet pass hole shall be located in each section on the top center line of the pipe about 3 ft. uphill from the field roundabout joint. Pass holes shall be tapped and provided with brass plugs.

7. *Marking:*

(*a*) The sections of the penstock, together with all special material, shall be carefully marked for identification in the field, in accordance with an erection diagram to be furnished by the Contractor, for field use.

(*b*) Each shop-assembled length shall be plainly marked at its lower end inside and on top outside with its number clearly painted. Two clear and distinguishable center punch witness marks shall be placed on the top outside of each length to identify corresponding rivet holes. The same rivet holes shall be further distinguishable by two clean paint marks.

(*B*) WORKMANSHIP:

8. *General:*

All workmanship shall be first class and in accordance with the best American shop practice. All pieces of the same mark shall be interchangeable. All sections of pipe, except taper pieces, shall be true circles of the required internal diameters.

9. *Shearing:*

Shearing shall be neatly and accurately done, and all portions of the work exposed to view shall be neatly finished. The cuts shall be clean, without drawn or ragged edges and without splitting away from the sheared edge.

10. *Planing:*

Edges of plates forming longitudinal seams for butt strap pipe shall be planed to bring the pipe to exact diameter and to insure tightness of the joints. The ends of all sections shall be properly cut to true lines.

11. *Beveling and Scarfing:*

(*A*) *Lap-Joint Pipe:*

In lap work, the edges of all plates must be properly cut or sheared to true lines and all edges which are to be calked in the finished pipe shall be properly beveled on a plane at approximately 70 degrees with the plane of the plate. At the end of each course where the lap of the longitudinal seam occurs, the plate must be reduced by planing or hammering, or both, to a fine edge to which one of the rivets of the round seam must be driven to insure tightness.

(*B*) *Butt Joint Pipe:*

(*a*) The edges of all butt straps shall be properly beveled for outside calking in the field.

(*b*) Roundabout seams of the shell plates shall not be beveled for inside calking, but the plates shall be cut square and so designed as to come within $\frac{1}{4}$ in. of each other.

(*c*) Wherever joints require scarfing, such shall be properly done to a fine edge and rivets properly spaced at such points to insure water-tightness.

12. *Punching and Reaming:*

(*a*) Punched holes shall be accurately spaced, true to line, so that when plates are brought together, holes shall exactly match.

(*b*) Only the sharpest dies and punches shall be used. The diameter of the die must never exceed the diameter of the punch by more than $\frac{3}{32}$ of an inch.

(*c*) The use of drift pins will be permitted only for drawing the material together. No drifting to enlarge unfair holes will be allowed. Necessary corrections shall be made with a reamer. Poor matching of punched holes will be sufficient cause for rejection.

(*d*) All rivet holes for butt-joint pipe and for lap-joint pipe where the plate thickness exceeds $\frac{3}{8}$ in. shall be sub-punched $\frac{1}{8}$ in. and reamed to size. The diameter of the finished rivet hole shall be $\frac{1}{16}$ in. greater than the diameter of the rivet as shown on the drawings.

(*e*) Rivet holes for lap-joint pipe with a plate thickness $\frac{3}{8}$ in. or less shall be punched to finished diameter without reaming.

13. *Rolling:*

All plates shall be bent cold, to a true circle of the specified diameter of the pipe, as nearly as practicable, by the use of a template.

14. *Drifting:*

No drifting to rectify unfair holes will be allowed. If holes require enlargement to admit the rivet or bolt, it must be reamed, and under no circumstances is the metal in the vicinity of the hole to be distorted or injured. The use of drift pins will be allowed only for bringing

together the several parts forming a member and they shall not be driven with such force as to injure the adjacent metal.

15. *Riveting:*

(*a*) The size of rivets called for on the drawings shall be understood to mean the actual size of the rivets before heating.

(*b*) Before riveting, all plates must be thoroughly cleaned and freed from rust and scale. Burrs shall be removed.

(*c*) Wherever possible, rivets shall be driven by pressure tools of sufficient capacity to upset the metal, exerting a slow and steady pressure of not less than fifty (50) tons for rivets of $\frac{3}{4}$-in. diameter nor less than seventy (70) tons for larger diameter, and retaining this pressure while the rivet head is being formed.

(*d*) All rivets, after driving, shall completely fill the hole, and have full heads concentric with the shank. No recupping nor calking of heads will be allowed. All loose, burned or otherwise defective rivets shall be cut out and replaced, great care being exercised not to injure the adjacent material, drilling out if necessary.

(*e*) In order to avoid shrinkage of the rivets on cooling, it will be required that the riveting pressures be held for the following periods of time on each rivet:

$1\frac{1}{4}$ in. diameter rivets	55 seconds
$1\frac{1}{8}$ in. diameter rivets	45 seconds
1 in. diameter rivets	35 seconds
$\frac{7}{8}$ in. diameter rivets	25 seconds
$\frac{3}{4}$ in. diameter rivets	25 seconds
$\frac{5}{8}$ in. diameter rivets	20 seconds
$\frac{1}{2}$ in. diameter rivets	18 seconds

All rivets shall be cone-head rivets.

16. *Calking:*

All seams must be calked on the outside in first-class boiler work fashion, and the inspection thereof completed before any coating is applied to the pipe.

The foregoing specifications (16. Calking) require outside calking; but recent practise seems to favor inside calking for heads up to about 200 ft. The advantages claimed for inside calking are that riveting can best be accomplished on the inside and it is an inviolable rule that calking must be done on the side on which rivets are driven. The advantages of inside riveting are as follows:

(*a*) The riveter requires more room than the backer-up, and, if riveting is done on the inside, less space is required between the bottom of the pipe and the surface of the excavated trench.

(*b*) The driven rivet heads are flatter and present less frictional resistance to the flow of water.

(*c*) The rivets can be heated and placed to better advantage from the outside.

The disadvantage of inside riveting is that, should leaks develop, they cannot be readily located, as the water may issue from the joint some distance from the point where it enters. However, those who favor inside rivet-

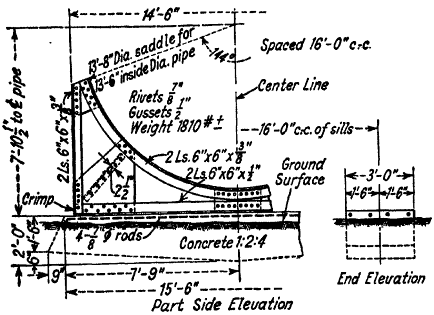


FIG. 293.—Saddle and Sill Details for 13-ft. 6-in. Diameter Steel Pipe.

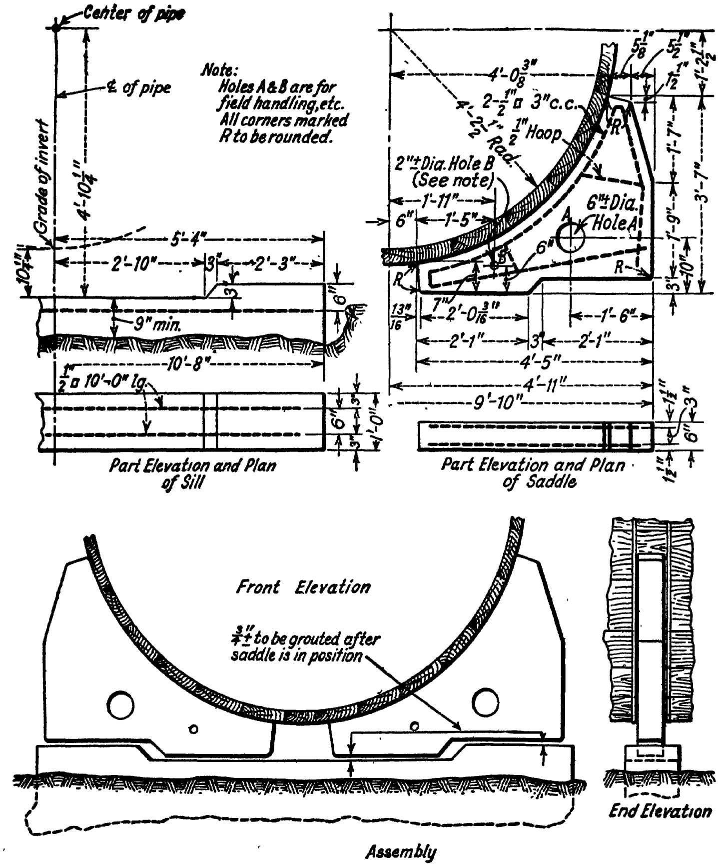


FIG. 294.—Concrete Saddle and Sill. Searsburg Development of New England Power Co.

For steep, exposed pipes, the supports are usually solid concrete piers of the type shown in Fig. 301. These piers are for rock foundations. For earth slopes as steep as those shown in Fig. 301, the pipe should be buried. No piers are usually provided for buried pipe except the temporary supports required while riveting is being done.

Changes in temperature will cause the pipe to expand and contract longitudinally if expansion joints are provided. Saddles and piers should be designed to withstand the force corresponding to frictional resistance to such movement, if the distance between the expansion joint and the anchorage is such that the movement will be appreciable. The contact between concrete piers or saddles and the pipe is often lubricated to reduce the frictional resistance. For very short pipe lines, concrete piers have been poured to embed the riveted joints of the pipe, but this practice is not recommended.

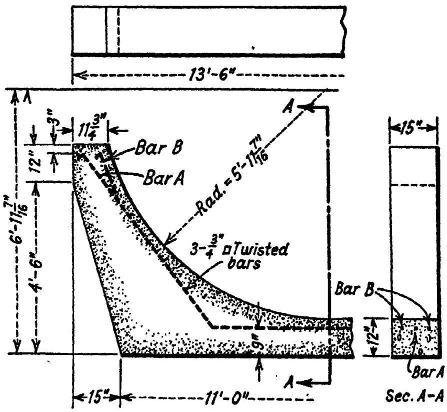


FIG. 295.—Reinforced Concrete Saddle for Wood-stave Pipe. Grace Development, Utah Power and Light Co. Concrete Sills Not Shown.

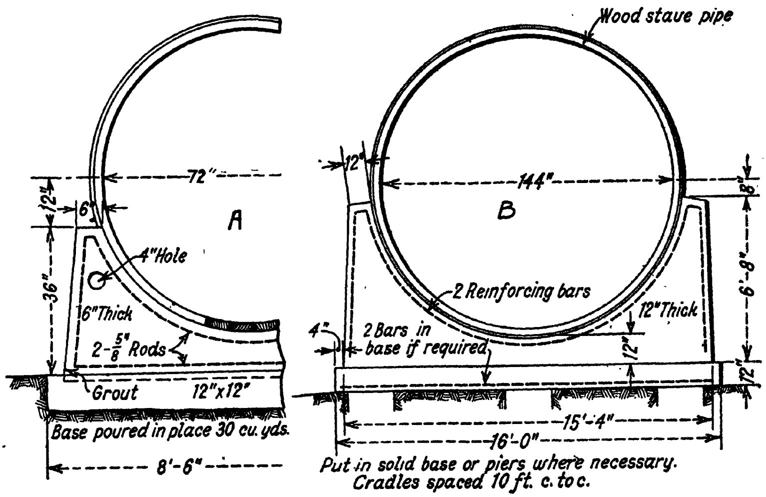


FIG. 296.—Typical Concrete Saddles. *A*—Pre-cast Cradle. *B*—Cradle Poured in Place. Continental Pipe Mfg. Co.

Opportunity is always present to space the joints and piers in such a way that they will not come into contact.

To reduce friction, or rather adhesion of the concrete to the pipe, a thin layer of tar paper has been placed between the concrete and the pipe; and, in one case, two layers of sheet packing, with a thin layer of graphite grease between, have been used. In some cases roller bearings have been used to reduce the friction. The use of a surface that will provide minimum friction is particularly advantageous where friction on the piers must be balanced by a concrete anchorage built in a difficult position.

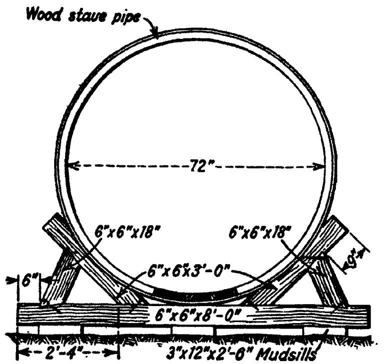


Fig. 297.—Typical Timber Cradle. Cradles Spaced 12 ft. C. to C. Continental Pipe Mfg. Co.

The spacing of piers and saddles is determined by the strength of the pipe, considered as a beam spanning between them, and the stress in the piers and saddles due to distortion of the pipe while being filled. A discussion of these features is given in Sec. 221.

There is no uniformity among designers with regard to the general dimensions of piers and saddles and their spacing. The angle of contact with the pipe varies from 60 to 180 degrees, usually from 100 to 180, and the spacing from 12 ft. to 40 ft. Unreinforced piers having a bearing of over 120 degrees have a tendency to crack. Angles of 60 degrees can only be used for small pipes with thick plates, as otherwise the pipe, if not very steep, will flatten out while being filled.

It is good practice to give the saddles a radius about 1 per cent larger than that of the outside of the pipe. Except when the pipe is under very low normal pressure, saddles are needed only to prevent excessive distortion while the pressure is being built up when filling; as the pipe, when under full pressure, is self-supporting. Consequently, for the larger-radius saddles, the pipe is allowed to be distorted somewhat, and this relieves the saddles of some load. Moreover, if the distortion of the lower part of the pipe is entirely restricted by the saddles when the pipe is being filled, while the upper part is comparatively free to undergo distortion, undue stress in the pipe (particularly undesirable in wood-stave pipe) will occur just above the top of the saddle.

For saddles of radius equal to that of the pipe and a large angle of contact, the pipe is not allowed to expand freely when under full head, and the saddle takes some of the stress. This feature accounts in a measure for the cracking of plain-concrete piers having a large angle of contact.

221. Design of Saddles and Circumferential Pipe Stiffeners.—If, owing to the use of thin plates, a steel pipe is not stiff enough to prevent undue distortion while filling, one of the following alternatives of design must be adopted.

1. The saddles may be placed close enough to provide the necessary stiffeners; or,
2. The saddles may be placed as far apart as beam action in the pipe will allow, and the pipe provided with circumferential stiffeners at regular intervals between the saddles.

Most steel pipes have been designed according to the first alternative, Table XLVI showing a few representative examples. However, it can be be shown that for many, if not most, cases, the second alternative is the most economical, particularly where the necessary piers and footings under the saddles average a considerable height. The following discussion,* prepared for this book by Mr. Ford Kurtz, is based on the second alternative of design.

The maximum safe span for steel pipes supported on saddles, and the corresponding loads on the saddles, are so intimately involved with practically every detail of the complete design of the pipe that any general rules as to the spans and saddle loads are, of necessity, restricted by assumptions that must be made as to other features of design, in order that these rules may be formulated.

Grade of Pipe.—The slope or grade of the pipe is the first of these general features. If the pipe is horizontal or practically so, the entire weight of water and pipe is carried by beam action and transmitted to the saddles as a loading normal to the surface of the saddle on which the pipe rests. If the pipe is on a slope making an angle α with the horizontal, the loading normal to the surface of the saddle on which the pipe rests is equal to the entire weight of water and pipe multiplied by cos α. The axial component of the entire weight, i.e., the component parallel to the axis of the pipe, equals the entire weight multiplied by sin α. The axial component of the weight of the pipe may be taken care of by friction between saddles and pipe, by longitudinal tension in the pipe due to transmitting this component upward along the slope to an anchorage, or by longitudinal compression in the pipe due to transmitting this component downward along the slope to an anchorage. The axial component of the weight of water may be taken care of by longitudinal tension in the pipe due to transmitting this component upward along the slope from an unanchored bend or a bulkhead to an anchor, or it may be taken care of by direct thrust into an anchor at the bend or bulkhead and thus produce no longitudinal stress in the pipe. The rules as to span and saddle loads are formulated for horizontal pipe, and the necessary changes to make them conform to the case of a sloping pipe are based on the following assumptions: (*a*) no friction between pipe and saddle; (*b*) no longitudinal stress due to weight of water in pipe; (*c*) axial component of weight of pipe transferred to anchorages by longitudinal compression in the pipe walls.

Allotment of Available Longitudinal Stress.—The plate steel commonly used in riveted pipe has an ultimate tensile strength between 55,000 and 65,000 lb. per square inch, with an average of 60,000 lb. per square inch, and an elastic limit strength of one-half the ultimate. From a consideration of numerous designs of circumferential joints for pipes of thicknesses ranging from ¼ to

* The remainder of this section.

TABLE XLVI

Examples of Steel Pipe Spans

Company	Los Angeles Aqueduct	Northern N. Y. Utilities, Inc.	Northern N. Y. Utilities, Inc.	Northern N. Y. Utilities, Inc.	Adirondack Pr. and Lt. Corp.	Connecticut Lt. and Pr. Co.	St. Lawrence Valley Pr. Corp.	Portland Electric Power Co.	Portland Electric Power Co.
Plant	San Francisquito P. H., No. 1	Eagle Falls	So. Edwards, No. 2	Browns Falls	Schaghticoke	Bulls Bridge	Sugar Island	Oak Grove	Oak Grove
Slope of pipe	1.0%	15.0%	1.0%	3.1%	0.77%	0.0%	0. 3%	2.8%	0.88%
Diameter	7′ 0″	9′ 0″	10′ 0″	11′ 0″	12′ 6″	13′ 0″	13′ 6″	9′ 0″	9′ 0″
Thickness	3/8″	3/8″	1/4″	15/32″	3/8″	1/2″	3/8″	9/32″	1/2″
Spacing of stiffeners	None	None	None	None	One midway between piers	None	None	7′ 9″	None
Size of stiffeners	Double-riveted butt. Joints, 6′ 0″ centers for stiffness	None	None	None	3″×5″×3/8″ L.	None	None	3″×5″×3/8″ Ls. †	None
Piers or Saddles: Material	Concrete piers with C. I. saddles	Concrete	Directly on ground	Steel	Concrete	Concrete and steel	Steel	Concrete and steel	Steel
Grip *	2′ 7″	1′ 0″	2′ 6″	8″	1′ 3″ above	1′ 7″	2′ 1″	0′ 0″ for steel 1′ 0″ for concrete	0′ 0″
Spacing	22′ 6″	16′ 0″	Continuous	16′ 0″	12′ 0″	....	16′ 0″	23′ 3″	22′ 6¼″
Clear span	About 22′ 0″	14′ 8″	Continuous	15′ 3″	10′ 0″	16′ 0″	15′ 0″	22′ 6½″	21′ 10½″
Performance when filling	Satisfactory	Satisfactory	Vert. diam. decreased 7.5″ when empty and 11.0″ when filling. Satisfactory	Satisfactory	Satisfactory	Satisfactory	Vert. diam. decreased 9″ when empty and 17.5″ when filling. Slight buckeling at saddles. Not satisfactory	Satisfactory	Satisfactory

* The vertical distance from center line of pipe down to top of pier or saddle.

† These between piers. There were also two 2½″×3½″×5/16″ stiffeners at each pier.

$1\frac{1}{2}$ in. it has been assumed that a mimimum circumferential joint efficiency of 55 per cent can be obtained. On these bases the following allotment of allowable longitudinal pipe stress has been adopted for the tables and diagrams hereinafter presented. It is obvious that, if the pipe is to be subjected to much less temperature or beam stress than is given in the following allotment, the allowable catenary stress may be increased, and *vice versa.* However, tables and diagrams for other stresses have not been included, and practical design will not necessitate their use except in extreme conditions.

Case 1. *Pipe Empty—Temporary Condition*

Rule.—Temperature stress must not exceed the minimum elastic limit strength at circumferential joints.

Minimum elastic limit strength, 27,500 lb. per square inch.

Minimum circumferential joint efficiency, 55 per cent.

Minimum gross longitudinal stress allowable, $0.55 \times 27{,}500 = 15{,}125$ lb. per square inch.

Allowable temperature stress (80° F. temp. change) 15,125 lb. per sq. in.

Case 2. *Pipe Just Full—Temporary Condition During Filling*

Rule.—Temperature stress plus catenary stress between stiffeners plus beam stress must not exceed 80 per cent of the minimum elastic limit strength at circumferential joints. (Factor of safety 2.5.)

	Lb. per sq. in.
Temperature stress (20° F. temp. change)	3,770
Catenary stress	7,830
Beam stress	500
Minimum gross longitudinal allowable stress $0.80 \times 27{,}500 \times 0.55$	12,100

Case 3. *Pipe Full and under Normal Operating Head*

Rule.—Temperature stress plus catenary stress between stiffeners plus beam stress must not exceed 50 per cent of the average elastic limit strength at circumferential joints. (Factor of safety 4.)

	Lb. per sq. in.
Temperature stress (20° F. temp. change)	3,770
Catenary stress	3,980
Beam stress	500

Average elastic limit strength 30,000.

Minimum circumferential joint efficiency 55 per cent.

Minimum gross longitudinal allowable stress $0.50 \times 30{,}000 \times 0.55$ 8,250

Temperature Stress.—The foregoing temperature stresses are based on the following assumptions as to temperature range of the pipe walls and temperature of pipe walls at which closing sections of pipe are riveted into place.

Assumed temperature of closure, plus 60° F.
Assumed maximum temperature, pipe full plus 80° F.
Assumed minimum temperature, pipe full, plus 40° F.
Assumed maximum temperature, pipe empty, plus 140° F.
Assumed minimum temperature, pipe empty, minus 20° F.

Catenary Stress between Stiffeners.—When a steel pipe supported on saddles is partly filled with water or is filled and under only a light pressure, there are produced in the shell large bending moments which tend to distort the pipe from its circular cross-sectional shape. If these moments produce unsafe bending stresses, stiffeners must be placed around the pipe to reduce these stresses to safe values. The effect of these stiffeners is to cause a portion of the weight of water between them to be carried by longitudinal catenary action of the pipe walls to the stiffener, where the bending moment corresponding to this portion of the load is imposed on the stiffener. Thus, *fundamentally*, stiffeners are provided for the purpose of reducing stresses in the pipe walls due to these distortive bending moments; but the size and spacing of stiffeners must be such that not only will the bending stresses in the pipe walls be kept within safe limits, but that also the longitudinal catenary stresses in the pipe walls, due to transmitting the necessary portion of the water load to the stiffeners, will be kept within the maximum limits already set.

For horizontal pipe, the maximum distortive moment due to the water pressure occurs at the bottom of the pipe when it is approximately half full, and produces tension on the outside of the pipe shell; the moment amounts to

$$M = \frac{r^3 y}{10}, \quad \ldots \ldots \ldots \quad (136A)$$

where M = the moment in inch-pounds per linear inch of pipe;
r = radius of pipe in inches, and
y = weight of water in pounds per cubic inch.

The maximum distortive moment due to the weight of the pipe itself also occurs at the bottom of the pipe and produces tension on the outside of the pipe shell; the moment amounts to

$$M_1 = \frac{r^2 w}{12}, \quad \ldots \ldots \ldots \quad (136B)$$

Where r = radius of pipe in inches, and
$w = \frac{W}{2\pi r}$, W being the average weight of pipe per linear inch of pipe line, including all excess weight due to lap of joints, rivets, overweight of plates, etc., i.e., w is the average weight per square inch of pipe shell including lap, etc.

The section modulus of the pipe shell per linear inch of pipe being $\frac{1}{6}t^2$ where t = thickness of pipe shell in inches, the maximum tension in the shell due to

these distortive moments can readily be found. When the pipe is just full of water the maximum moment due to water pressure is reduced to $\frac{1}{24}r^2y$; but it then occurs at four points, viz., the ends of the horizontal and vertical diameters. When the pipe is under a head of water sufficient to produce a gross tensile stress of p lb. per square inch in the pipe walls, the maximum moment due to water pressure still occurs at the same four points, but is reduced in magnitude to

$$\frac{\frac{1}{24}r^2y}{1+\frac{3}{2}\cdot\frac{p}{E}\cdot\frac{r^2}{t^2}}, \quad \ldots \ldots \ldots \quad (136C)$$

where E = modulus of elasticity of steel in pipe walls in pounds per square inch.

It is apparent from the foregoing that if a value of 20,000 lb. per square inch is adopted as the safe tensile stress for the condition of maximum moment during filling of the pipe, the tension will be only slightly over 8000 lb. per square inch when the pipe is full and under no pressure, and considerably less than this as the pipe rounds out under pressure. The filling condition with pipe about half full therefore fixes the lower limit of thickness for unstiffened pipe of various diameters, according to Eqs. 136A and 136B and, on the basis of a stress of 20,000 lb. per square inch and an excess pipe weight of 23 per cent over theoretical, gives the following limiting values:

TABLE XLVII

Diameter of Pipe in Feet	Minimum Thickness of Plate for Unstiffened Pipe in Inches *	Diameter of Pipe in Feet	Minimum Thickness of Plate for Unstiffened Pipe in Inches*
4	0.12	13	0.75
5	0.17	14	0.83
6	0.23	15	0.93
7	0.29	16	1.02
8	0.36	17	1.12
9	0.43	18	1.22
10	0.50	19	1.33
11	0.58	20	1.43
12	0.66		

* Unstiffened pipe is pipe without the stiffening action of saddles, relatively close together, or circumferential stiffening angles. Thinner plate may be used if the saddles are not too far apart (see Table XLVI).

If the thickness of the pipe shell, as determined by the strength of longitudinal joints and the maximum head on the pipe, or by the minimum allowable thickness to take care of corrosion, rigidity during construction, etc., is less than that given in the Table XLVII for various diameters, then the pipe should be provided with stiffeners, for, otherwise, the supporting saddles would have to provide the necessary stiffening action, which is not their function, and their spacing would be fixed by stiffening requirements instead of by other and more proper considerations. In other words, if no stiffeners were provided, the spacing of the saddles would have to be too close for maximum economy.

When a pipe is provided with stiffeners, it is difficult to formulate the true expression for the division between that portion of the distortive moment of the water load which is resisted by the pipe shell and that portion which is resisted by the stiffeners; but it is believed that the following expression can safely be used for all purposes of design:

$$\frac{1-x}{\sqrt[3]{x}} = \frac{R^{4/3}(Et^4 + \frac{3}{2}pr^2t^2)}{10.89r^6}, \quad \ldots \ldots \quad (136D)$$

where x = the fractional part of the load between stiffeners, the distortive moment of which is resisted by the stiffeners;

E = modulus of elasticity of steel in pipe walls, in pounds per square inch;

t = thickness of pipe walls, in inches;

p = gross tension in pipe walls due to head of water measured at center of pipe, in pounds per square inch;

r = radius of pipe, in inches;

s = distance center to center of stiffeners, in inches;

$R = \frac{rs}{t}$ = stiffener spacing in terms of $\frac{t}{r}$;

$$\frac{R}{144} = \frac{\left(\frac{r}{12}\right)\left(\frac{s}{12}\right)}{t} = \frac{(r \text{ in feet})(s \text{ in feet})}{(t \text{ in inches})} \quad \ldots \quad (136E)$$

On the basis of the foregoing equation for x, Tables XLVIII and XLIX have been prepared to show the maximum allowable spacing of stiffeners consistent with the allotted catenary stresses before given.

If the value of p under normal operating head is greater than the value given in Table XLIX, use values of $\frac{R}{144}$ from that table; but if it is less, then use values of $\frac{R}{144}$ from Table XLVIII.

TABLE XLVIII

MAXIMUM ALLOWABLE VALUES OF $\frac{R}{144}$ FOR $p = 0$ UNDER NORMAL OPERATING HEAD

Diam. in Feet	Thickness of Pipe in Inches																
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{7}{16}$
7	242	242															
8	52	242	242														
9	47	242	242	242													
10	46	48	242	242	242												
11	...	46	49	242	242	242											
12	...	46	47	50	242	242	242										
13	...	45	46	47	50	242	242	242	242								
14	...	45	45	46	48	50	242	242	242	242							
15	...	...	45	46	46	48	50	242	242	242	242	242					
16	...	...	45	45	46	47	48	51	242	242	242	242	242				
17	...	...	45	45	45	46	47	48	50	242	242	242	242	242			
18	...	...	...	45	45	45	46	47	48	50	53	242	242	242	242		
19	...	...	...	45	45	45	46	46	47	48	50	52	242	242	242	242	
20	...	...	...	45	45	45	45	46	46	47	48	50	52	242	242	242	242

TABLE XLIX

MAXIMUM ALLOWABLE VALUES OF $\frac{R}{144}$ FOR THE STATED, OR GREATER, VALUES OF p UNDER NORMAL OPERATING HEAD

Diam. in Feet		Thickness of Pipe in Inches																
		$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{7}{16}$
8	$\frac{R}{144}$	242																
	p	100																
9	$\frac{R}{144}$	242																
	p	600																
10	$\frac{R}{144}$	141	142															
	p	1700	1000															
11	$\frac{R}{144}$	....	154	242														
	p	....	1600	400														
12	$\frac{R}{144}$	....	137	175	242													
	p	....	2700	1300	350													
13	$\frac{R}{144}$	....	130	143	230	242												
	p	....	4100	2500	900	300												
14	$\frac{R}{144}$	....	126	134	149	242	242											
	p	....	5800	3700	2,200	800	300											
15	$\frac{R}{144}$	....	....	128	137	155	242	242										
	p	....	....	5100	3,400	2100	800	100										
16	$\frac{R}{144}$	....	....	126	131	140	160	242	242									
	p	....	....	6900	4800	3200	2000	800	400									
17	$\frac{R}{144}$	....	....	124	127	133	143	169	242	242								
	p	....	....	8900	6,400	4600	3200	2000	800	400								
18.	$\frac{R}{144}$	....	....	....	125	129	135	146	172	242	242							
	p	....	....	....	8,100	6100	4300	3100	2000	900	500							
19	$\frac{R}{144}$	....	....	....	124	126	131	137	148	175	242	242	242					
	p	....	....	....	10,200	7700	5700	4300	2600	1900	1000	500	200					
20	$\frac{R}{144}$	....	....	....	123	125	128	132	138	150	175	218	242	242				
	p	....	....	....	12,600	9600	7300	5700	4300	3000	2100	1200	600	200				

All stiffeners between saddles should be designed to take care of the bending moment due to the portion of the water load defined by the value of x from Eq. (136D) and under the half-full condition; i.e., the moment in the stiffener derived from Eqs. (136A) and (136B) will be:

$$M = \frac{xr^3ys}{10} + \frac{xr^2ws}{12} \quad \text{. . . . .} \quad (136F)$$

The value of x will be found from Eq. (136D), it being remembered that for this case $p = 0$. Since this is a temporary condition, a value of 18,000 to

20,000 lb. per square inch can be used for the safe stress in the stiffener, depending somewhat upon the type of stiffener used.

If the above-given values of $\frac{R}{144}$ are not exceeded, the bending stresses in the pipe shell will at all times be within safe values, even when the pipe is under its maximum head, *if* the efficiency of longitudinal joints does *not exceed* 90 per cent and *if* the longitudinal joints are located midway between horizontal and vertical center lines of the pipe section in order to avoid the points of maximum bending stress in the pipe shell.

There must, of course, be stiffeners at the supports or saddles; but as these take care not only of distortive moments due to weight of pipe and water but also of the moments caused by carrying the beam shear down into the saddle, their design will be considered under the subject of "Saddle Loadings."

A numerical example, explanatory of the foregoing theory, is given hereafter.

Modifications for Sloping Pipe.—If the axis of the pipe makes an angle α with the horizontal, all bending moments shown by Eqs. (136*A*, *B*, *C*, and *F*) for a horizontal pipe should be multiplied by $\cos \alpha$. This will result in lower minimum thicknesses of plate for unstiffened pipe, values slightly in excess of the true ones being obtained by multiplying those in Table XLVII by $\sqrt{\cos \alpha}$. In Eq. (136*D*) for the determination of x, the right-hand member should be divided by $\sqrt[3]{\cos^2 \alpha}$. All of the values of $\frac{R}{144}$ in Tables XLVIII and XLIX can conservatively be increased by dividing these by $\cos \alpha$, and values of p in Table XLIX will tend to decrease, so that it is conservative to use the values corresponding to a horizontal pipe as given.

Beam Stress.—The pipe having been properly stiffened between supports and over the saddles, if necessary, and the allowable beam stress limited to 500 lb. per square inch for stiffened pipe and to about 2000 lb. per square inch for unstiffened pipe, to take care of the catenary action which is inevitable even between saddles, the fixing of allowable spans resolves itself into a simple problem of beams. In what follows, the pipe is assumed to be a continuous beam, so that the case of a pipe with a circumferential joint or joints having zero longitudinal strength (such as an expansion joint) is not covered. In addition to this, the assumption is made that the length of sections or courses of pipe is such that no circumferential joint is closer than 4 ft. to the center of any saddle. This is equivalent to assuming 8-ft. courses of pipe with saddles located at middle of courses. This assumption results in having the maximum moment over the support come where there is no circumferential joint and materially increase the allowable spans. Just as in the stiffener design, the excess weight of pipe is taken as 23 per cent of the theoretical weight.

On the basis of the foregoing assumption, curves of maximum spans for horizontal stiffened pipe are shown in Fig. 298. For pipe having shell thicknesses greater than the minimum values specified in Table XLVII for unstiffened pipe, the maximum safe span will generally be limited by designs of stiffeners for the pipe at the cradles and by designs of the cradles themselves,

rather than by allowable beam stresses in the pipe. It is suggested that for such pipes the maximum safe span be so determined, with the general reservation, however, that in no case should the span exceed 50 ft. for any diameter of pipe or size of plate.

If the axis of the pipe makes an angle α with the horizontal, all bending moments and stresses due to beam action will be multiplied by $\cos\alpha$, and the allowable span for any set of conditions will be divided by $\sqrt{\cos\alpha}$, making it larger. All other rules remain the same, including the limitation of the maximum span of 50 ft.

Numerical Example No. 1.—Assume a pipe 16 ft. in diameter on a 30-degree slope, with no expansion joints and all bends anchored to take hydraulic and temperature thrusts. Maximum head, 160 ft. Normal operating head, 140 ft. Longitudinal joint efficiency, 88.9 per cent. Circumferential joint

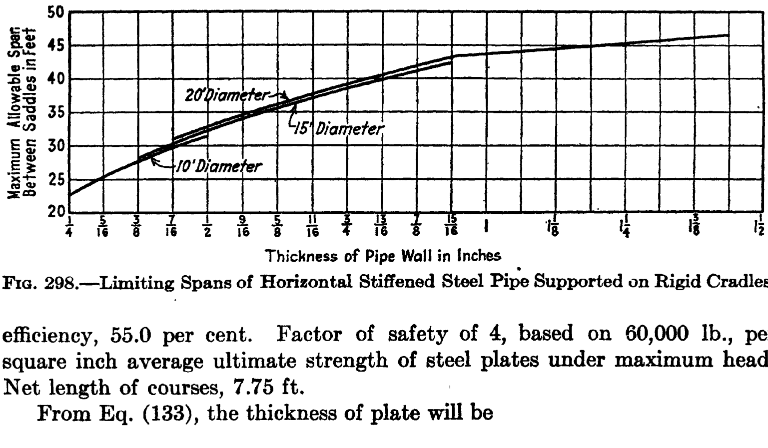


FIG. 298.—Limiting Spans of Horizontal Stiffened Steel Pipe Supported on Rigid Cradles.

efficiency, 55.0 per cent. Factor of safety of 4, based on 60,000 lb., per square inch average ultimate strength of steel plates under maximum head. Net length of courses, 7.75 ft.

From Eq. (133), the thickness of plate will be

$$t = \frac{2.6 \times 160 \times 16}{15{,}000 \times 0.889} = 0.5 \text{ in.}$$

From Table XLVII, modified as explained for a sloping pipe, it is seen that the pipe should be stiffened, the limiting thickness being $1.02\sqrt{\cos 30^\circ} = 0.95$ in. for a 16-ft. pipe. The design of the stiffeners follows:

From Eq. (133), the gross tension in the pipe plates at normal operating head is

$$p = \frac{2.6 \times 140 \times 16}{0.5} = 11{,}667 \text{ lb. per square inch.}$$

From Table XLIX, it is found that, for a diameter of 16 ft. and a thickness of 0.5 in., the foregoing value of p is greater than the value of 3200 given in the table. Hence $\frac{R}{144}$, taken from this table, is 140.

From Eq. (136*E*),

$$\frac{R}{144} = 140 = \frac{8s}{0.5}.$$

Therefore, s = maximum allowed stiffener spacing = 8.75 ft. for a horizontal pipe. For the sloping pipe the maximum spacing is $\frac{8.75}{\cos 30°} = 10.10$ ft. and one stiffener per course, or 7.75 ft., will be the adopted stiffener spacing.

The value of x must next be determined. From Eq. (136*E*) for a stiffener spacing of 7.75 ft.,

$$R = \frac{8 \times 7.75 \times 144}{0.5} = 17{,}856.$$

Then, from Eq. (136*D*), modified as explained for a sloping pipe and for $p = 0$ under the conditions of pipe half full,

$$\frac{1 - x}{\sqrt[3]{x}} = \frac{17{,}856^{4/3} \times 29{,}000{,}000 \times 0.5^4}{10.89 \times 96^6 \times \sqrt[3]{\cos^2 30°}} = 0.1092,$$

and

$$x = 0.895.$$

Then, from Eq. (136*F*), modified as explained for sloping pipe, we have

$$M = \frac{0.895 \times 8^3 \times 62.5 \times 7.75 \times \cos 30°}{10}$$
$$+ \frac{0.895 \times 96^2 \times 0.142 \times 1.23 \times 7.75 \times 12 \times \cos 30°}{12}$$
$$= (230{,}800 + 9700) \text{ in.-lb.} = 240{,}500 \text{ in.-lb.}$$

based on the assumption of 23 per cent for excess weight of lap, etc.

At a safe stress of 20,000 lb. per square inch in the stiffener angle during filling of pipe, the required section modulus would be 12 in.^{3} If 6 in. of plate is considered as acting with a stiffener angle, this section modulus could be provided by 2 – $6 \times 3\frac{1}{2} \times \frac{9}{16}$ in. angles with 6-in. legs outstanding, or by some other structural shape, such as a channel. If two angles are used per course, the logical arrangement would be to make the spacing 3.88 ft. instead of having two angles close together at the center of a course. The problem of stiffener design becomes a purely structural one, once the maximum moment is known; *but in all cases* the reinforcing ring must be a complete ring around the pipe, securely riveted to the pipe. Splices in stiffener rings should be located over the longitudinal joints of the pipe shell, i.e., midway between horizontal and vertical center lines of the pipe section, in order to avoid the points of maximum bending stress in the stiffener ring; but the splices should nevertheless be liberally designed for bending moment. The maximum safe span for horizontal pipe, from Fig. 298, is about 32 ft., and for a slope of 30 degrees this increases to $\frac{32}{\sqrt{\cos \alpha}}$, or 34.3 ft. This span should be a multiple of the net course length, and hence 31 ft. should be adopted.

Numerical Example No. 2.—Same pipe as in Example No. 1, except that the maximum head is 100 ft. and the normal operating head is 55 ft. Preceding as in the former example, we have:

From Eq. (133),

$$t = \frac{2.6 \times 100 \times 16}{15{,}000 \times 0.889} = 0.3125 = \tfrac{5}{16} \text{ in.}$$

But the omission of thicknesses less than $\frac{3}{8}$ in. (0.375) for a 16-ft. pipe in Table XLIX indicates that thicknesses less than 0.375 in. are not recommended, for various practical reasons. Hence a thickness of 0.375 in. will be adopted.

Table XLVII shows that a 16-ft. pipe having a thickness less than $1.02 \sqrt{\cos 30^\circ} = 0.95$ should be stiffened.

From Eq. (133), the gross tension in the pipe plates at normal operating head is

$$p = \frac{2.6 \times 55 \times 16}{0.375} = 6100 \text{ lb. per square inch.}$$

From Table XLIX, it is found that, for a diameter of 16 ft. and a thickness of 0.375 in., the foregoing value of p is *less* than the value of 6900 given in the table. Hence, as previously explained, the value of $\frac{R}{144}$ must be taken from Table XLVIII or $\frac{R}{144} = 45$.

From Eq. (136*E*),

$$\frac{R}{144} = 45 = \frac{8s}{0.375}.$$

Therefore, s = maximum allowed stiffener spacing = 2.11 ft. for a horizontal pipe. For the sloping pipe, the maximum spacing is $\frac{2.11}{\cos 30^\circ} = 2.44$ ft.

This would mean our stiffener rings per course, which is an excessive number. If, now, the thickness of plate is increased to $\frac{7}{16}$ in. (0.4375), we have

$$p = \frac{2.6 \times 55 \times 16}{0.4375} = 5230 \text{ lb. per square inch,}$$

which is *greater* than the value of 4800 given in Table XLIX and hence $\frac{R}{144} = 131$ can be taken from that table.

From Eq. (136*E*),

$$\frac{R}{144} = 131 = \frac{8s}{0.4375}.$$

Or,

$$s = 7.16 \text{ for a horizontal pipe,}$$

and

$$\frac{7.16}{\cos 30^\circ} = 8.27 \text{ ft.}$$

Hence it would be more economical to use the thicker plate and increase the spacing of the stiffeners.

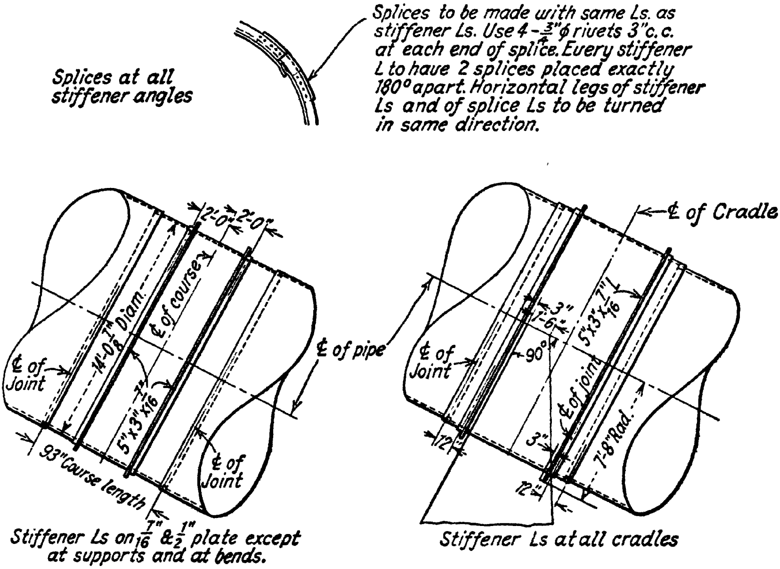


FIG. 298A.—Details of Stiffener Angles.

Loadings at Saddles and Saddle Stiffeners.—The determination of loading on the saddles and of the moment in the pipe stiffeners at the saddles is based on the assumptions that the saddle is practically rigid as compared with the pipe, that the pressure between pipe and saddle is everywhere normal to the surface of contact, i.e., radial as referred to the pipe, and that the portions of the pipe and stiffeners, if used, above the saddle act as a fixed-end, circular arch to transmit loads to the saddle. The saddle loadings are of two kinds, concentrated and distributed, as indicated in Fig. 299. The concentrated loads are due to arch thrust and occur at the upper ends of the vertical legs of the saddles; the distributed loads occur over the entire surface of contact between pipe and saddle. The maximum bending moment in the stiffeners, or in the pipe if stiffeners are not used, occurs at the top of the vertical legs of the saddles.

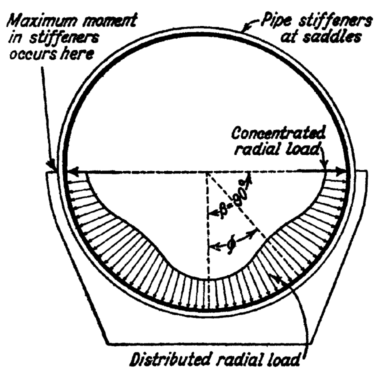


FIG. 299.—Diagram of Saddle Loadings.

For a horizontal stiffened pipe the following loadings will obtain, the concentrated loads being expressed in pounds at the top of each saddle leg; the distributed loads, in pounds per linear inch of saddle circumference; and the stiffener moments or moments in the pipe shell if stiffeners are not used, in inch-pounds.

β = *Half Angle Subtended by Saddle* = 90 *Degrees.*

Concentrated radial load

$$= (0.135l + 0.185s)(ry + 2w)r. \qquad (136G)$$

Distributed radial load

$$= \left(2\cos\phi - \tfrac{4}{3}\cos^3\phi + \frac{\pi}{4}\right)(ry + 2w)l - (\cos\phi - \tfrac{4}{3}\cos^3\phi)(ry + 2w)s. \qquad (136H)$$

Maximum bending moment in stiffeners

$$= (0.008l + 0.047s)(ry + 2w)r^2, \qquad (136I)$$

where l = distance center to center of saddles, in inches;

s = distance center to center of stiffeners, in inches (for an unstiffened pipe, place s equal to zero);

r = radius of pipe, in inches;

y = weight of water, in pounds per cubic inch;

$w = \dfrac{W}{2\pi r}$;

W = average weight of pipe, in pounds per linear inch of pipe, including all excess weight due to lap of joints, rivets, overweight of plates, etc.;

ϕ = an angle measured from the lower end of the vertical center line of the pipe section.

It should be noted that the maximum allowable span of saddles may in some cases be dictated by the design of stiffeners at the supports, to take care of the maximum bending moment, as well as by the design of saddles and foundations for the heavy loadings. Therefore, Fig. 298 of maximum safe spans must be considered as referring to beam action only, and in every case the economy of stiffener and saddle design must be investigated before deciding on the proper span to adopt.

For unstiffened pipe it is only necessary to place s equal to zero in the foregoing expressions. If the axis of the pipe makes an angle α with the horizontal, all the foregoing loads and moments are to be multiplied by $\cos\alpha$.

If there is movement of pipe in saddle, frictional resistance will be developed, with resulting overturning moment on the saddle. This friction loading can easily be determined from the foregoing normal loadings if the coefficient of friction is known. Since it is costly to design a saddle to take care of large moment in the direction of the pipe axis, measures should be taken either to prevent the movement or to reduce the coefficient of friction.

All of the foregoing saddle loadings apply only to straight pipe. If the pipe is curved or has a horizontal angle in it between saddles, there will be additional and unsymmetrical loadings due to the hydraulic and temperature thrusts at curves. These loadings are difficult and uncertain of determination, and it is recommended that on curves the pipe be supported otherwise than by saddles.

222. Anchorages.—*Location.*—It is customary to install anchorages at angle points in the pipe and to provide expansion joints between the anchorages. If the pipe line is flat, the expansion joints are usually located midway between the anchorages, in order to reduce the movement of the pipe over the saddles or piers to a minimum. On steep slopes, however, the anchorage at the crest of the hill is frequently more difficult to install than the one at the foot of the hill. For this reason, it is advantageous, in such cases, to locate the expansion joint near the up-hill anchor, thus transferring all of the friction on the piers to the lower anchor. This arrangement also facilitates the erection of the pipe, as it is usually difficult, on steep pipe, to erect that portion between the expansion joint and the upper anchorage, on account of the tendency of the pipe to slide down hill. If the distance between anchorages is so great that the sum of the frictional resistance at the saddles or piers becomes greater than the anchorages or circumferential joints should take, additional expansion joints are provided. Additional expansion joints are also required if the distance between anchorages is so great that the movement of the pipe, due to extremes of temperature, is greater than a single expansion joint can accommodate. In such cases an anchorage should be provided between every two expansion joints, to equalize the movement of the pipe on the piers.

Anchorages and expansion joints are not usually provided for buried pipes, unless anchorages are necessary to support a steep pipe before covering, or unless the pipe is to be previously filled for testing.

Anchors should be designed so that they will take care of, by gravity alone, the resultant of all forces. Rods for anchorages should be used only in the hardest and most solid ledge rock. It is not safe to use them in ordinary rock.

Analysis of Stresses for Anchors.[6]—The following formulae and diagrams give a complete analysis of all the stresses acting upon an anchor. Many of these forces will generally be of small magnitude, but they have been included in this discussion in order to make it complete. As shown on the diagram, Fig. 300, all of these forces acting at the bend are finally combined into vertical and horizontal components, which, in turn, combined with the weight of the anchor itself, give a resultant that must lie within the middle third of the anchor base.

Definition of Symbols:

h = static head at any point;
A = inside area of pipe, in square inches;
p = pressure, in pounds per square inch, at any point;
a = angle of upstream leg with horizontal;

[6] From Report of Hydraulic Power Committee, National Electric Light Ass'n, 1923.

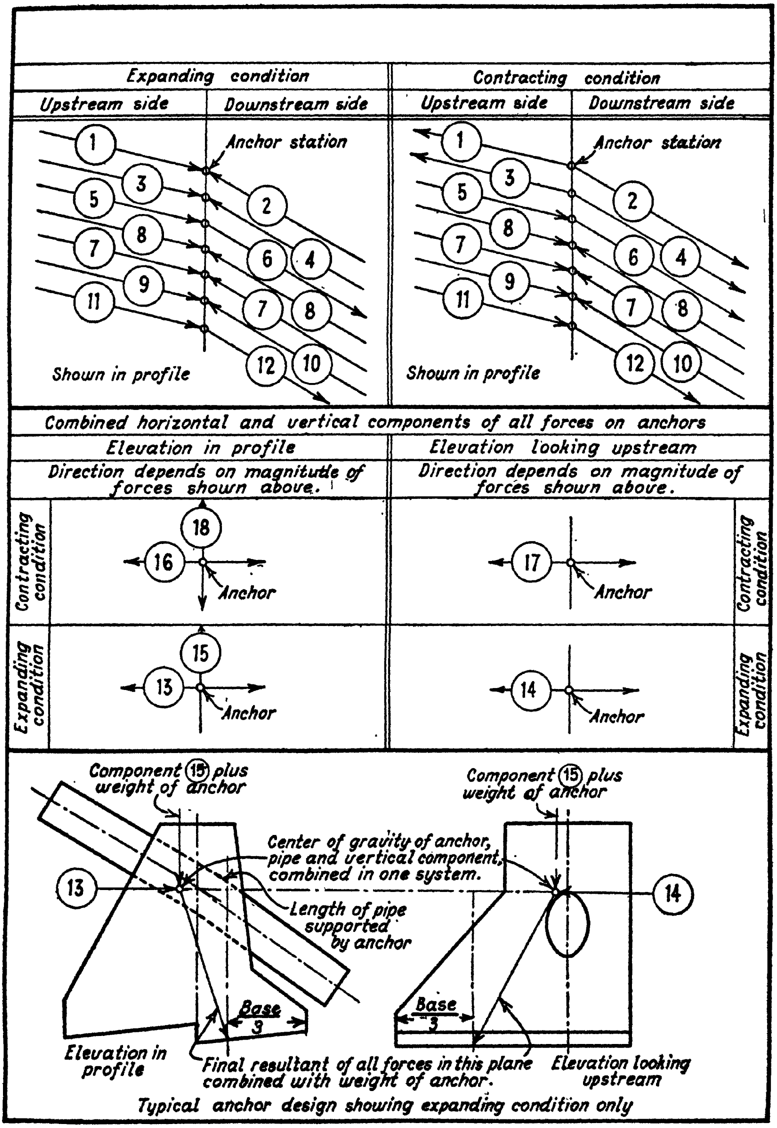


Fig. 300.—Diagrams Showing Exterior Forces on Anchors.

b = angle of downstream leg with horizontal;
c = exterior horizontal angle between center lines of upstream and downstream legs;
d = true deflection angle of bend;
W = weight of pipe from a point midway between anchor and first pier upstream to expansion joint;
W' = weight of pipe from a point midway between anchor and first pier downstream to expansion joint;
P = weight of water in pipe W;
P' = weight of water in pipe W';
Q = weight of water and pipe between points midway between adjacent piers above and below anchor;
f = friction coefficient of pipe on piers. (This should be 0.6 for untreated contacts as steel on steel or concrete, 0.4 to 0.5 for lubricated surfaces, and much less for roller or hinged bearings);
V = velocity of water, in feet per second;
C = circumference of pipe, in feet, at expansion joint above anchor;
C' = circumference of pipe at expansion joint below anchor;
S = weight of pipe from center line of anchor to point midway between anchor and adjacent upstream pier;
S' = weight of pipe from center line of anchor to point midway between anchor and adjacent downstream pier;
F = friction in expansion joint per linear foot;
E = area of exposed end of pipe at expansion joint, in square inches;
R = area of pipe in square feet;
w = weight of water, per cubic foot;
g = acceleration of gravity;
K = reduction in area at reducer above anchor;
K' = reduction in area at reducer below anchor.

Penstock Anchor Forces:

1. Friction on supports, due to expansion or contraction on upstream side of anchor $= f(W + P) \cos a$.
2. Friction on supports due to expansion or contraction on downstream side of anchor $= f(W' + P') \cos b$.
3. Friction of expansion joint on upstream side of anchor due to expansion or contraction $= CF$(F = 500 lb. per linear foot, determined by experiment).
4. Friction of expansion joint on downstream side of anchor due to expansion or contraction $= C'F$.
5. Force due to deadweight of pipe acting parallel to center line of pipe on upstream side of anchor $= (W + S) \sin a$.
6. Force due to deadweight of pipe acting parallel to center line of pipe on downstream side of anchor $= (W' + S') \sin b$.
7. Force due to bend in pipe with water flowing $= RVwV/g$.
8. Hydrostatic force at anchor $= pA$ (acts along axis of pipe toward bend on each side of anchor).

9. Force on exposed end of expansion joint, upstream side of anchor $= Ep$.
10. Force on exposed end of expansion joint, downstream side of anchor $= E' p$.
11. Force due to reducer above anchor $= pK$.
12. Force due to reducer below anchor $= pK'$.
13. Horizontal component in plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
14. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, expanding condition.
15. Vertical component for algebraic sum of all forces on anchor, expanding condition.
16. Horizontal component in plane of upper penstock center line for algebraic sum of all forces acting on anchor, contracting condition.
17. Horizontal component normal to plane of upper penstock center line for algebraic sum of all forces on anchor, contracting condition.
18. Vertical component for algebraic sum of all forces on anchor, contracting condition.

Figure 301 gives details of a typical anchor. Where anchors are located on tangents and at the intake, several circumferential angles, riveted or welded to the pipe, serve to transfer the forces from the pipe to the anchors. Anchors at sharp bends seldom need this provision, as the anchorage should envelop all or part of the bend.

223. Expansion Joints.—Some means are necessary for taking care of expansion and contraction, due to changes in temperature, in exposed steel pipes. A discussion of the relative location of expansion joints and anchorages is given in Sec. 222.

Short pipes with frequent bends are sometimes considered flexible enough for this purpose; but usually some form of expansion joint is necessary.

The use of expansion joints for buried pipe is seldom necessary, as such pipe is not exposed to great extremes of temperature. However, if a buried pipe is solidly retained at each end, has no bends to accommodate movement, and has inadequate circumferential rivets, the omission of an expansion joint may cause failure.

Expansion joints eliminate excessive longitudinal stress in the pipe and thus permit of the use of lighter circumferential joints.

There are two general types of expansion joints, i.e., the slip joint and the diaphragm joint. Fig. 302 is a slip joint of the type frequently used on welded pipe lines. This particular joint has one surface copper-plated, to obviate the possibility of its rusting tight.

Figure 303 is a typical slip joint, built up of riveted steel pipe.

The packing of the stuffing box of slip joints can be satisfactorily done with square braided hemp or flax packing impregnated with graphite. Joints of this type, when properly packed, give entire satisfaction and have insignificant leakage. They require some attention, however, and are thought by some to be inferior, under certain conditions, to the diaphragm joint. The conditions favorable to diaphragm joints are slight movement and low

head. A typical diaphragm joint is shown in Fig. 304. The movement of the pipe is provided for by the flexibility of the steel in the joint. For cold climates, the plates, A, should not extend across the opening, because circula-

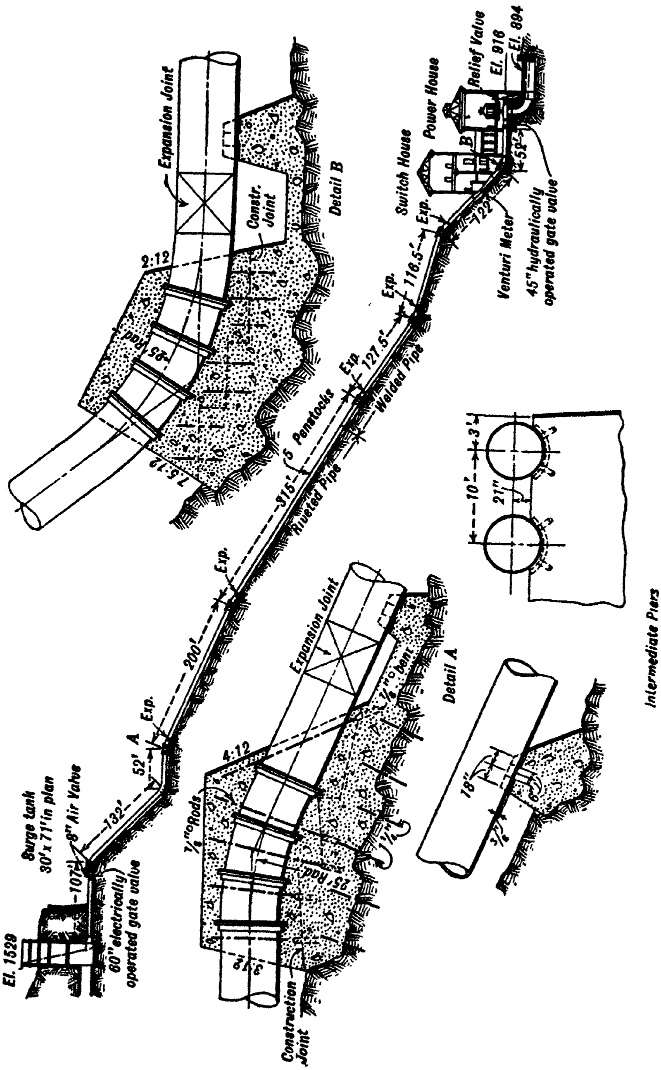


FIG. 301.—Anchors and Piers for the Tallulah Falls Development. Georgia Railway and Power Co. Engineering News-Record, Vol. 69, p. 361.

tion of the water inside the joint should be provided to prevent freezing. In very cold climates the expansion joint should be boxed in to prevent freezing, as described in Sec. 225. Such joints are not particularly well

adapted for heads greater than about 60 ft., because, if they possess the stiffness required to withstand internal pressure under greater heads, they are

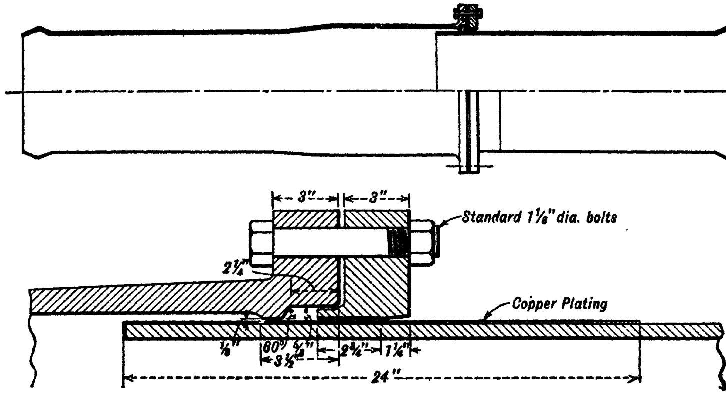


FIG. 302.—Welded Expansion Joint with Electroplated Copper Sleeve. M. W. Kellogg Company.

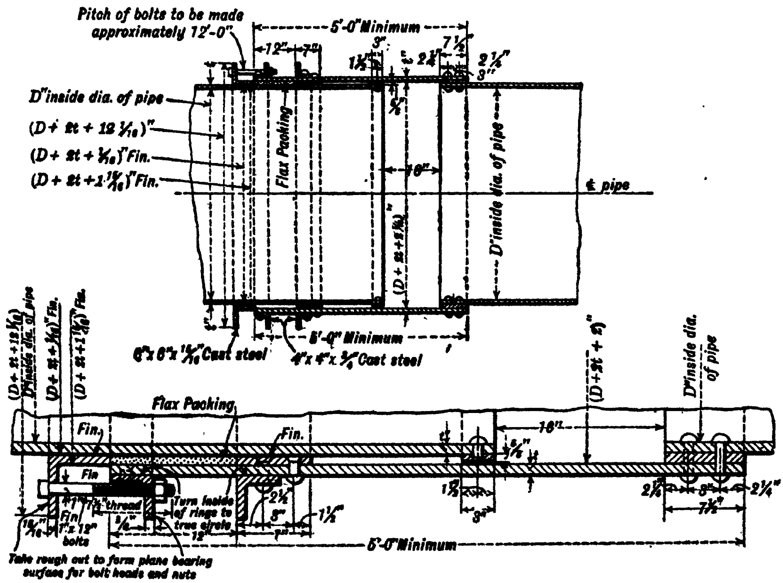


FIG. 303.—Type of Slip Joint for Riveted Steel Pipe Line, Pacific Gas and Electric Co.*

* From Report of Hydraulic Power Committee, National Electric Light Association, 1923.

not flexible enough for expansion joints. Movement in such joints is not as great as that of the slip type of joint, and hence a greater number are required for a given length of pipe.

The expansion and contraction of steel penstocks, due to temperature changes, is greatest when the penstock is empty. As during the construction period the penstock is likely to remain empty through a wide range of temperature changes, expansion joints should be designed to take care of these conditions. On the Catskill aqueduct a number of determinations were made of the actual expansion in steel pipe lines, due to temperature changes. Table L shows that the actual expansion agrees closely with the computed

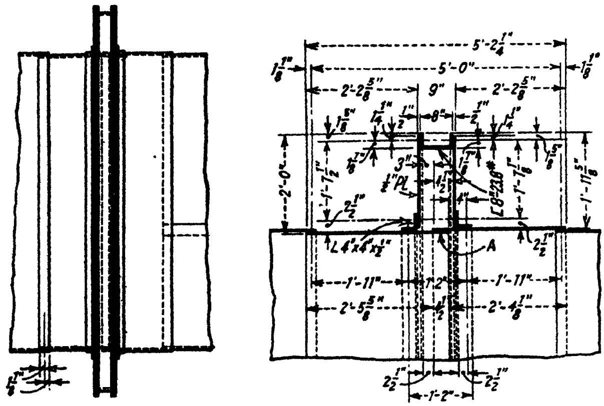


FIG. 304.—Diaphragm Expansion Joint at Rock Island Development, Tennessee Power Co., Manufactured by M. H. Treadwell Co.

theoretical expansion. The theoretical expansion was computed with a coefficient of 0.0000072, and the temperature used was the mean atmospheric temperature, inside and outside the pipe.

TABLE L

TEMPERATURE CHANGES IN STEEL PIPE—PIPES EMPTY AND EXPOSED *

Diameter, Feet and Inches	Length, Feet	Temperature Change, Degrees, Fahrenheit †	Expansion—Inches	
			Observed	Theoretical ‡
8–10	3078	15	3	3½
8–10	3078	20	4–5	4½
8–10	3078	25	6–7	6
8–10	3078	30	7	7¼
8–6	807	19	1¼	1¼
8–6	807	30	2⅛	2
8–6	1292	15	1¾	1⅝
8–6	1292	33	3⅝	3⅝

* From "Building Additional Siphons of Catskill Aequeduct," Mansfield Merriman, Eng. News-Record, Vol. 90, p. 869.
† Mean of atmospheric temperature, inside and outside of pipe.
‡ Theoretical expansion computed with coefficient of 0.0000072.

224. Painting Steel Pipes.—Numerous paints for steel pipes are on the market, but none of them give a permanent protection to the steel. The life of a steel pipe above ground can be prolonged indefinitely by frequent painting. The outside surface of a buried pipe is inaccessible except at a great expense. In each case it would be economical to pay several times the present market price for the best paint, if a permanent protective coating were available.

It is not possible to give here the relative merits of the different paints that are on the market. It must be stated, however, that under no circumstances should the quality of paint for steel pipe be sacrificed for a saving in cost.

The most rigid inspection of painting should be made. The best paint poorly applied is no better than a paint of very inferior quality, and frequently not as good.

The paint to be adopted depends upon the location of the surface to be covered, i.e.,

(*a*) the outside of the pipe above ground,
(*b*) the outside of the pipe under ground,
(*c*) the outside of the pipe in dark, damp compartments,
(*d*) the inside of the pipe.

Each of these divisions is subject to different paint-disintegrating influences, and usually requires a different class of coating. For important pipe lines, the paint should be purchased only from those manufacturers who have had long experience, unless an exhaustive test of the paint is to be made.

For buried pipe, care should be taken that the refill does not contain tannic acid from decayed vegetable growth, or other ingredients which may affect the durability of the paint. The soil should be subjected to rigid chemical analysis, by the engineer and by the chemists of the paint manufacturer.

Of late years it has been the practice to give the first coat to the pipe after it has been delivered at the site, because a shop coat frequently becomes scraped off in many places during transit. The small amount of corrosion occurring during short transit can be easily brushed off before painting in the field.

225. Protection against Freezing.—In very cold climates, an ice sheet will form on the inside of exposed steel pipes unless the pipe is short or the velocity high. This ice sheet may become sufficiently thick to effect the hydraulic properties of the pipe and, during a thaw, it frequently breaks away and enters the turbine in sufficient quantity to plug the gates. Water hammer in such cases may be considerably greater than that computed for normal operation. Whether or not trouble will be experienced from ice depends upon the following factors:

(*a*) the velocity during off-load periods;
(*b*) the duration of such periods;
(*c*) the climate at the site;
(*d*) the exposure of the pipe to prevailing winds;
(*e*) the depth of the forebay;
(*f*) the depth of the pipe entrance below water surface.

Equations have been derived to show the minimum velocity to prevent freezing; [7] but such equations are unreliable as the velocity to prevent freezing depends upon the temperature of the water as it enters the pipe, which is not known exactly. Moreover, a very small difference in the assumed water temperature results in an enormous difference in the computed velocity to prevent freezing.

Therefore the probability of ice troubles must be estimated according to experience and comparison with existing plants in the vicinity. Pipes over 1000 ft. long, in climates similar to that of the mountainous regions of northern New York, operating at usual velocities and at about 50 per cent load factor on a general power and lighting load, are likely to give occasional trouble unless protected.

Pipes may be protected from freezing by burying them or covering them with wood or concrete housing.

There is practically no danger from freezing in a pipe covered with 3 ft. of earth, even though the frost line is twice this distance below the surface, provided the center line of the pipe is below the frost line.

The 9-ft. by 550-ft. steel penstocks of the No. 1 Plant at Shawenegan Falls, Canada, where ice troubles were encountered, were provided with wooden coverings consisting of 1-in. planks supported on ribs and covered with one layer of tar paper. This covering eliminated ice troubles. The penstocks of the No. 2 Development were provided with a more permanent covering consisting of a 2-in. thickness of gunite on a steel framework. An air space was provided between the gunite and the steel shell, and a thick layer of tar paper was used for the inside form of the gunite.[8]

At the Grand Falls Development of the Anglo-Newfoundland Development Company at Grand Falls, Newfoundland, where ice troubles were serious enough to break the turbine runners frequently, the penstocks were completely housed in with a light wooden structure covered with 3-ply asphaltic roofing. No attempt was made to make a frost-proof housing, other than to provide a more or less dead air space. The results were very satisfactory.

Additional data on ice troubles in pipe lines are furnished by the Northern New York Utilities, Inc. The plants are located about 44 degrees north latitude, in New York State. All pipes are exposed. Elevations are above sea level.

Browns Falls, Elevation 1340.—This plant has 3550 ft. of 12.5-ft. wood-stave pipe line. At the lower end of the wood-stave pipe line there are 2850 ft. of 11-ft. steel pipe, at the end of which is the vertical riser of the surge tank. From this point two steel penstocks, each 186 ft. long and 8 ft. in diameter, lead to the power house. Full plant discharge is 900 sec.-ft. The load factor is low and the plant shuts down frequently over Sunday. There has been no trouble from ice.

Eagle Falls, Elevation 1420.—This plant has a steel pipe line 2900 ft. long and 9 ft. in diameter. There is no surge tank. Full plant discharge with

[7] Such an equation is given in *Eng. Record*, Vol. 69, p. 362.

[8] See "Guniting Steel Penstocks" to Eliminate Trouble Due to Ice Formation, by J. A. McCrory, The Canadian Engineer, Jan. 9, 1913.

three units is 476 sec.-ft. and the usual minimum plant discharge is 85 sec.-ft. The plant is frequently shut down over Sunday. The velocity in the pipe line for full load has been about 7.5 ft. per second, and for the usual minimum load about 1.3 ft. per second. On several occasions this plant has been shut down because of ice troubles in the pipe line.

South Edwards No. 2, Eelvation 840.—This plant has a 10-ft. diameter steel pipe line about 1500 ft. long with a surge tank at the lower end, located at the side of the pipe and connected to it by means of an off-set. Full plant discharge is 510 sec.-ft. and the plant is frequently shut down over Sunday. There have been no troubles due to ice in the pipe line.

In one instance a novel method was used to protect the pipe from ice troubles. Holes were drilled in the top of the pipe and the water was allowed to spray the top of the pipe to form an insulating ice sheet on the outside.

226. Buried vs. Exposed Pipe.—For various reasons, steel pipe should not be buried unless local conditions render it necessary.

The conditions favorable to buried pipe are as follows:

(*a*) Pipes extending down steep hillsides on earth foundation make anchoring and supporting very difficult. If the pipe is buried, anchors and supports are not usually necessary.
(*b*) On steep side-hill locations there is frequently danger from land slides, snowslides, and falling rocks, which would injure the pipe unless buried.
(*c*) In cold climates, where the pipe is very long and the velocity low, it is sometimes less expensive to bury it than to provide other means of protection against freezing.
(*d*) Where the pipe passes through earth cut, it is often cheaper to bury it with the excavated earth than to provide cradles and sills.
(*e*) Expansion joints can be reduced or eliminated for buried pipes; but this feature is usually an insignificant influence.
(*f*) Very small pipes can frequently be supported by burying at less cost than by providing cradles or piers.

The advantages of exposed pipe are as follows:

(*a*) More room is provided for construction.
(*b*) Such pipes are more accessible for inspection, maintenance, and repairs. This is the predominating feature favorable to exposed pipe.
(*c*) Where pipe is buried, inspection and renewal of paint on the outside of the pipe is not likely to occur frequently, and buried pipe may therefore be considered to have a shorter life.
(*d*) Exposed pipe is less expensive to install if the material is difficult to excavate.

Pipe that is to be buried is generally supported in the trench temporarily, on blocking. In refilling the trench, the material should be carefully tamped into place as the material under the bottom of the pipe is very likely to settle, leaving a space of several inches under the pipe.

Experience has shown that the use of porous material, allowing free drainage, results in longer life of the pipe than the use of impervious clays. The chemical characteristics of the soil surrounding the pipe should be carefully studied, as certain soils are very detrimental to most pipe coatings. (See Sec. 224.)

Pipe that is only partly buried usually corrodes quickly at or just below the ground line, owing to the extreme variation in temperature and moisture.

227. Adjuncts to Steel Pipe.—The necessary adjuncts to steel pipe lines that are common to other types of conduits are described in Chapter XVII.

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CHAPTER XXI

WOOD-STAVE PIPE

By Byron E. White

229. General.—A general outline of the purpose and use of wood-stave pipe in hydro-electric practice, and a discussion of those features of wood-stave pipe which are common to all conduits are given in Chapter XVII. The pipe between the forebay and the surge tank is commonly called the " pipe line " and that between the surge tank and the turbines is called the " penstock." Unless the contrary is specifically stated, the term " pipe " as used herein is intended to apply to both penstocks and pipe lines.

For successful operation, the size of wood-stave pipe may vary between rather wide limits; but there is usually one size which will make for the greatest economy of design. The reader is referred to Sec. 87 for a general discussion of economic design and to Sec. 183 for its application to the special case of conduits.

230. Types of Wood-stave Pipe.—Wood-stave pipe consists of a shell formed of wooden staves bound together by steel bands. The bands are designed to withstand the bursting pressure of the water within the pipe. There are two general types of wood-stave pipe, designated, according to the method of manufacture or assembly, as (1) machine-banded wood-stave pipe, and (2) continuous wood-stave pipe.

Machine-banded Wood-stave Pipe.—Machine-banded wood-stave pipe is a manufactured product and is transported to the job as a finished pipe in lengths from 3 to 20 ft. as desired. It is regularly made in sizes from 2 to 32 in. in diameter and for heads up to 500 ft. It has been made 42 in. in diameter for use under low pressures.

Machine-banded wood-stave pipe is suitable only for the conveyance of relatively small quantities of water, on account of its restricted size. It can be made to withstand heads up to 500 ft. or more in the small sizes. This type of pipe has been used very extensively for water mains for domestic water supply in the West, and also for conveying water for irrigation, hydraulic sluicing, etc., as well as for sewers and drains.

Continuous Wood-stave Pipe.—Continuous wood-stave pipe is built up in place where it is to be used. The lumber is delivered in the form of staves, together with the steel bands and other accessories.

One of the particular advantages of continuous wood-stave pipe is the relatively small size and weight of the individual parts, which permits the transportation of single staves and bands to the site where the pipe is to be erected.

This is a special advantage in rough, mountainous, or wooded country where transportation is difficult and material may have to be carried to the site by pack animals or men.

Continuous wood-stave pipe has been constructed in diameters from 6 in. up to 16 ft. Some of the pipe manufacturers are prepared to quote on pipes up to 20 ft. or more in diameter. The maximum heads possible for this type are shown in Fig. 308 for pipes from 3 ft. to 22 ft. in diameter.

Continuous wood-stave pipe, on account of its flexibility, ease and rapidity of construction, comparative ease of transportation, and its relatively low cost at lower heads, as contrasted with other types of pipe, has a distinct field of usefulness in connection with hydro-electric power stations. Within certain limitations of head, the cheapness of wood-stave pipe as compared with other pipes makes it particularly suitable for power developments where it is essential to keep capital expenditures down to a minimum.

Relative Costs.—The fields of continuous and machine-banded pipe overlap somewhat, and the choice between them is usually made on the basis of ease and cost of transportation. For instance, on account of its greater bulk, transportation costs for the larger sizes of machine-banded pipe are much greater than for continuous pipe where long freight hauls are involved. For the same reason, there are also many cases where continuous pipe material may be transported to the site of the work much more easily than machine-banded pipe sections in the larger sizes.

231. Machine-banded Wood-stave Pipe.—A typical machine-banded wood-stave pipe is shown in Fig. 305. The staves are assembled on a form, in a machine in which they are wound spirally with galvanized round steel wire, or a galvanized flat steel band, which is fastened to the wood by a special clip at one end of the pipe section, wound on under the proper tension, and again fastened with a similar clip at the other end of the section of pipe, and cut off. The two ends of the section are then turned down square, if separate collars are to be used; or a recess is formed at one end and a tenon on the other end. After this the section is rolled through a bed of sawdust, which absorbs the excess asphaltum, leaving the pipe in such condition that it can be handled after drying.

Machine-banded wood-stave pipe is made up in random lengths, which may be from 3 or 4 ft. to 20 ft. long, in order to make the best use of the available lumber of suitable quality.

The end joints between adjacent lengths are made in two ways: (1) by means of separate collars encircling the ends of adjacent sections of pipes (Fig. 305), and (2) by means of a recess and tenon or slip joint (Fig. 306).

The use of the slip-joint or "inserted-joint," type (Fig. 306), of pipe is restricted to low-head work and to cases where a slight amount of leakage is not objectionable, as, for instance, irrigation work, hydraulic sluicing, etc. The maximum head to which this type of pipe is suited is not far from 100 ft.

In the case of the design having square ends and collars, the hoops or wire banding is left off for a suitable distance from each end, and adjacent sections are connected together by means of wood-stave, steel, or cast-iron collars, which are made to fit snugly around the ends of the adjacent sections. The

wood-stave collars are made 6 to 8 in. in length, and in the smaller sizes, up to about 12 in. in diameter, are machine-banded, in the same manner as the pipe. Collars 6 in. in length are used for diameters up to 12 in. For the larger sizes, one-piece hoops or bands, provided with shoes, nuts, and washers, are used for tightening up the collars. Not less than three bands should be used per coupling, and these require " cinching " or tightening after the pipe has been erected.

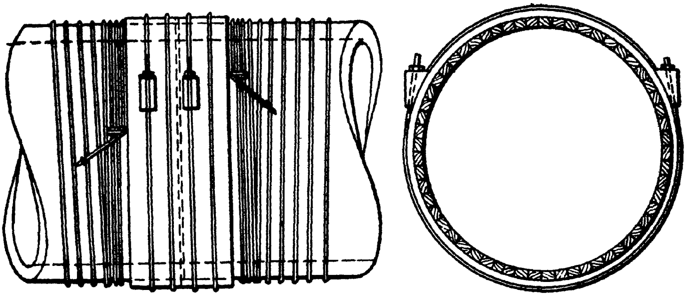

FIG. 305.—Typical Banded Collar for Wire-wound Pipe.

Wood-stave collars should always be creosoted as they are not subject to complete saturation from the water within the pipe, and many cases may be cited where untreated collars have failed from decay.

In the recess-and-tenon type of pipe, the tenons are made of such dimensions as to fit tightly into the recessed ends, and each section is assembled with the recessed end outward, and driven home by means of a wooden follower and ram.

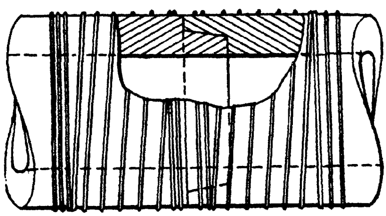

FIG. 306.—Recessed or Tenon Type Wire-wound Pipe Joint.

This type of wood-stave pipe, unless of large diameter and under low heads, or laid in unstable ground, does not usually require special supports. It may be assembled directly in the bottom of the trench, or on the ground, and may be laid to curves of rather long radius, depending on the shortness of the sections and the diameter.

The following tables give the approximate ranges of size, weight, and wire banding, for this type of pipe, as made by a prominent manufacturer:

TABLE LI

RANGE OF SIZES AND THICKNESS OF STAVES FOR MACHINE-BANDED WOOD-STAVE PIPE

Inside Diameter, Inches	Thickness of Staves, Inches	For Heads Less Than Feet	Thickness of Staves, Inches	For Heads Over, Feet
2	1			
3	1			
4	$1\frac{1}{16}$			
5	$1\frac{1}{16}$			
6	$1\frac{1}{8}$			
8	$1\frac{1}{8}$			
10	$1\frac{1}{8}$	250	$1\frac{1}{4}$	250
12	$1\frac{3}{16}$	250	$1\frac{1}{4}$	250
14	$1\frac{3}{16}$	250	$1\frac{1}{4}$	250
16	$1\frac{1}{4}$	200	$1\frac{5}{16}$	200
18	$1\frac{1}{4}$	200	$1\frac{5}{16}$	200
20	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
22	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
24	$1\frac{5}{16}$	150	$1\frac{3}{8}$	150
26	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
28	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
30	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150
32	$1\frac{3}{8}$	150	$1\frac{1}{2}$	150

NOTES.—Maximum spacing of wire, 3 in. Working stress of wire, 15,000 lb. per square inch. Maximum pressure head, 500 lb. approx. Range of lengths of pipe sections, 6 ft. to 25 ft.

TABLE LII

WEIGHTS OF UNTREATED MACHINE-BANDED COUPLING TYPE WOOD-STAVE PIPE, AND SIZES AND SPACING OF WIRE

Size, Inches	50-foot Head			400-foot Head		
	Weight, Pounds	Foot-size, Number	Wire-spacing, Inches	Weight, Pounds	Foot-size, Number	Wire-spacing, Inch
2	3.3	8	3	4.1	8	$\frac{3}{4}$
4	5.7	8	3	7.3	8	$\frac{11}{16}$
6	8.5	6	3	11.6	6	$\frac{11}{16}$
8	10.8	6	3	15.1	6	$\frac{5}{8}$
10	13.7	4	3	21.3	4	$\frac{11}{16}$
12	16.8	4	3	27.0	4	$\frac{9}{16}$
14	19.6	4	3	34.3	2	$\frac{5}{8}$
16	24.1	2	3	41.7	2	$\frac{9}{16}$
18	26.8	2	3	48.7	2	$\frac{1}{2}$
20	30.6	2	3	58.4	2	$\frac{7}{16}$
22	33.6	2	3	64.3	1	$\frac{1}{2}$
24	36.4	2	3	74.3	1	$\frac{7}{16}$

232. Continuous Wood-stave Pipe.—Continuous wood-stave pipe is assembled, stave by stave, by either of two methods: (1) It may be assembled in the permanent cradles in which it is to rest. (2) In the absence of fully semi-circular permanent cradles, temporary portable wooden cradles are used for setting up the lower half of the pipe. After this the top half is laid up on two collapsible, portable wooden forms in the interior of the pipe, which, with the portable cradles, are moved forward for setting up successive lengths.

In the usual method of construction, staves of approximately uniform length are used for each section, with alternate staves projecting from 2 to 3 ft. beyond the adjacent ones. The staves of the succeeding section are driven into the spaces between these projecting staves, thus tying the pipe together and providing continuous beam action between supports. This method of

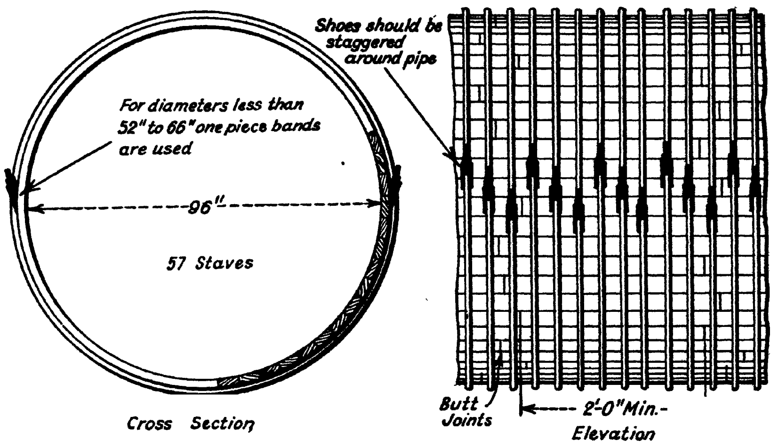

FIG. 307.

assembly is shown in Fig. 307. Continuous pipe is sometimes assembled with staves of random lengths, no attempt being made to keep the ends of alternate staves approximately together, longitudinally. The only precaution necessary to observe is that there should be at least a 2-ft. break in all longitudinal joints; the break, of course, may be of any greater length.

The staves are bound together by circular bands of steel, of such cross-sectional area, and so spaced along the pipe, as to resist most efficiently the bursting pressure within.

In setting up the pipe, all bands finally required over a support are put in place, together with sufficient intermediate bands, approximately 2 ft. apart on centers, to hold the pipe together. When a suitable length of pipe has been set up, the remaining bands are placed and all are cinched up, preparatory to filling the pipe with water.

This type of pipe derives its strength against bursting from the steel bands which encircle the pipe and by which it is made tight. The ability of the pipe to stand up without collapsing, when empty, depends on the thickness

of the staves and the strength of the bands; and it has been found in practice that certain thicknesses of staves are most suitable for certain pipe diameters, heads, and conditions. These matters will be further discussed in succeeding sections.

Owing to the flexibility of the staves, it is possible to bend continuous wood pipe to fit vertical, horizontal, and inclined curves, within certain limits which are specified in Sec. 238. Hence, it is possible within these limits to construct such pipe continuously without the introduction of special bends.

The pipe is made tight by "cinching," or tightening down the nuts on the bands. In erection, it is an advantage to have the staves as dry as possible, as in that case little or no further drying out from the effects of sun and wind will take place after the pipe has been erected. If the staves are not dry enough, constant watchfulness is required and the bands must be tightened up whenever the pipe shows signs of weakness from drying out. After the pipe has been completed and tied in to other structures, filling with water should take place slowly, as, in spite of all precautions, it is impossible to cinch up the bands tightly enough to prevent some leaks, and it is desirable that such leaks should take place under as little hydrostatic head as possible, so as to limit the amount of water leaking out of the pipe. Excessive leakage might cause erosion and endanger the safety of the pipe.

A further reason for slowly filling with water is to reduce the total stress in the bands during the time that the saturation of the staves is taking place with the consequent swelling of the wood and temporary increase in band tension, as described in Sec. 237.

As soon as the water comes in contact with the wood, some water is absorbed, causing the staves to swell slightly. This action usually closes practically all leaks within a few hours. Those leaks which do not close quickly, or which do not close after the pipe has been re-cinched, have to be dealt with by means of wedges or plugs.

Where a small leak exists between adjacent staves, a carpenter's chisel is driven in about ½ in. from the edge of one of the staves, and a very thin, wide wedge of soft wood, usually of white pine, if obtainable, is driven into the opening thus made, forcing the edges of the staves tightly together. This is sufficient, in general, to close all but the most serious leaks. The wedge should be cut off flush with the outer surface of the pipe. In some cases, it is necessary to drive soft-wood plugs into small holes where wedging in the above manner is not sufficient. Wedges should never be driven into the longitudinal seams, as they tend to spread the staves apart and extend the leak.

The sections that follow pertain principally to continuous wood-stave pipe and to the special requirements in its design, installation, and maintenance. Much of the matter is equally applicable to machine-banded pipe.

From the information at hand, it appears that the design and installation of wood-stave pipe has in the past been largely done by the exercise of judgment and the use of rule-of-thumb methods rather than by the application of theory. On some important points, little information of an engineering character has been published. From the experience of many years with successful

and unsuccessful installations, certain principles and practices which approximate standards, have been evolved. The observance of these will result in a safe and conservatively designed installation.

The data in regard to design have been collected from the best available sources and represent conservative present practise. Several design curves were developed from such information in order to present it in more usable and rational form.

233. Staves.—*Cross-section of Staves.*—Staves are formed in the mill so that the interior and exterior surfaces are arcs of circles, the interior having a radius requal to one-half of the nominal pipe diameter. The object in making the exterior surface of the staves truly circular is to distribute the bearing stresses between the bands and the staves equally around the circumference, and to take full advantage of the band strength and the bearing value of the wood. A polygonal exterior surface, which would result if plane-outer-surface staves were used, would concentrate the bearing stresses at and near the edges of staves, reducing the maximum head that might be impressed on the pipe and also inducing checks in the wood. The edges of the staves are customarily made radial so that adjacent staves may make full contact with each other throughout their entire surfaces.

Some engineers have specified that one side of each stave should be milled to a plain radial surface, with a bead $\frac{1}{16}$ in. or $\frac{1}{8}$ in. high on the other radial side. There is considerable doubt as to the advantage of the beaded stave, as it is thought by some that the crushing of the bead may possibly hasten decay.

The plain radial joint, when properly installed and cinched up, has been found to make a satisfactorily tight pipe.

Thickness and Width of Staves.—The dimensions of staves depend principally upon the following requirements:

1. Staves should have sufficient structural strength to resist the tendency to go "out of round" when filled with water, and to bridge properly between cradles without sagging.

2. Staves should be thin enough, within the limits imposed upon them, to be thoroughly saturated when under working head or pressure, if they are not treated with preservative.

3. The width of stave is dictated purely by considerations of economy and the use of stock material.

A compromise has to be effected between requirements 1 and 2, as they lead in opposite directions.

The thicknesses of staves shown in Fig. 308 have been found from experience to be suitable for the designated sizes of pipe and heads, under varying conditions. There is at least one formula in existence which indicates much smaller thicknesses, but experience indicates that a strong, satisfactory pipe requires the use of the thicknesses herein referred to.

Figure 309 gives the weight of staves for continuous wood-stave pipe per linear foot of various diameters from 1 ft. to 22 ft., for untreated Douglas fir. Douglas fir has a weight of approximately 36 lb. per cubic foot; redwood, approximately 26 lb. per cubic foot; cypress, 30 lb. per cubic foot; and long

leaf yellow pine, approximately 44 lb. per cubic foot, depending upon the degree of dryness. As the specifications usually call for 8 lb. of creosote oil per cubic foot, a proportionate increase in weight per foot is to be allowed for creosoted pipe.

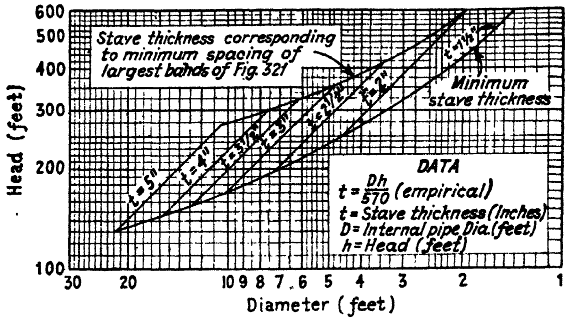

FIG. 308.—Diagram Showing the Thickness of Staves for Wood-stave Pipe Used in Present Practice.

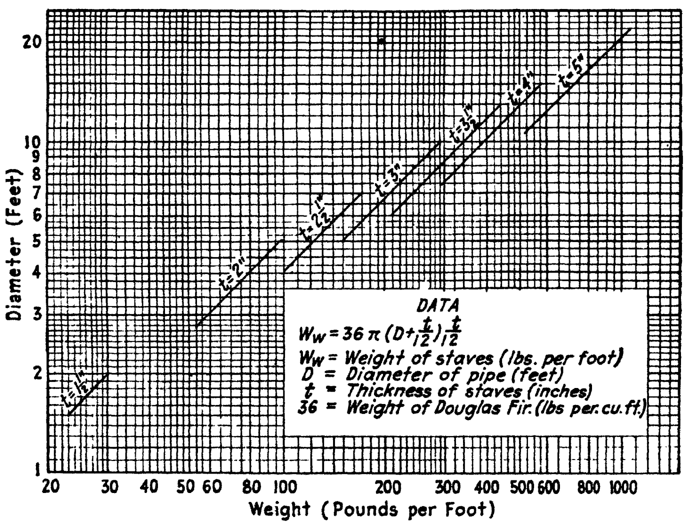

FIG. 309.—Diagram Showing the Weight of Douglas Fir Staves in Wood-stave Pipe per Foot of Length. For Redwood Multiply by 0.72.

In the manufacture of staves, selected lumber is required, and, in addition, it is most economical to use stock sizes of lumber; that is to say, sizes such as 2 × 4s, 3 × 4s, 4 × 6s, etc., so that the rejections can be used commercially for other purposes. There is an economical width of stave for each diameter, which in general may be approximately stated as follows:

For pipes up to 24 in. in diameter, the width should not be greater than 4 in.

For diameters of 30 in. up to 14 ft., the stock should be 6 in. wide.

For greater diameters, stock not more than 8 in. in width should be used.

Staves as wide as 12 in. have been used in some instances but are wasteful of lumber unless left with flat interior and exterior surfaces, in which case it is impossible to get proper bearing between bands and staves and thus to prevent the development of cracks and checks.

Wood for Staves.—Many varieties of timber, such as hemlock, spruce, yellow pine, Douglas fir, redwood, and cypress, have been used for the staves of wood pipe, and satisfactory pipes have been made from all of these timbers.

The woods most commonly used for pipes in recent years are redwood and Douglas fir, which grow in the Pacific Coast States. Spruce and yellow pine of suitable quality are difficult to obtain in large enough quantities; while hemlock and other soft woods are becoming scarce and also contain defects which make them relatively unsuitable for long-life pipes, as compared with pipes made of redwood and fir.

Although there is some dispute between the advocates of redwood and Douglas fir, the evidence seems to indicate that, for an untreated pipe, redwood may give a somewhat longer life than fir, under certain conditions of soil, climate, and use. With proper creosote or other treatment, however, it is probable that either pipe when properly designed, installed, and maintained, will give satisfactory service for many years. Failures of pipes made from both woods have occurred in cases where proper precautions in design, installation, maintenance, and use were not observed.

Wood for staves should be practically straight-grained, and free from shakes, and sapwood should be excluded from the exterior of the pipe. A small amount of sapwood in the interior, since it will be continuously saturated, is not considered a disadvantage, especially if creosoted. No sapwood should be allowed on the exterior. A limited number of small, live knots are permissible, but should not be allowed close to ends of staves. (See specifications.)

The growth rings should preferably be nearly parallel to the circumference of the pipe. Where they are diagonal, extending through from the interior to the exterior surfaces of the stave, splitting sometimes results under high heads, and in very porous specimens of lumber excessive percolation of water may occur.

If the growth rings are practically parallel to the radii of the pipe, excessive percolation may take place through the soft rings under high heads.

Redwood has less mechanical strength than fir or long leaf yellow pine. Lower stresses, both in bearing and bending, should be used with redwood pipe. It is to be noted, however, that the bending strength in a wood pipe is not brought into play sufficiently in properly designed installations to make this a material factor, except under extremely high heads with thin staves and bands of large cross-sectional area.

It is generally agreed that air-dried lumber is preferable for wood pipe lines of redwood, which shows a considerable loss of strength when kiln-dried.

Douglas fir staves, however, if kiln-dried properly, i.e., not too rapidly, show practically no loss of strength, and it is standard practice to specify kiln-dried lumber.

234. Bands.—*Allowed Stresses in Bands.*—Methods for determining the loading or maximum internal pressure in closed conduits, including water hammer and surges, are given in Sec. 184. The usually accepted working stress for steel bands is 15,000 lb. per square inch. However, the working stress to be adopted depends upon the probable accuracy of the computed loading, the elastic limit of the steel used, the relative importance of the pipe, and the danger of damage to other important structures in case of failure.

Recommended values of working stresses in percentage of the elastic limit of the steel are given in Sec. 184.

Figure 321 shows the spacing between centers of bands from $\frac{3}{8}$ in. to $1\frac{1}{4}$ in. in cross-sectional diameter on pipes of 1 ft. to 22 ft. internal diameter, for various total static heads within the limits of safe stresses in, and maximum and minimum spacing between, the bands. This diagram is based on a working stress of 15,000 lb. per square inch. In using it, the sum of all elements that will simultaneously cause pressure within the pipe should be taken for the head. For instance, the sum of the static head and the head due to surge at the point under consideration should be taken, and not the static head only.

The stress in the bands, caused by swelling of the staves when first saturated with water, is treated in Sec. 237.

Where narrow structural steel cradles are used, a bending moment in the bands lying in the cradle exists at the top edge of the cradle, caused by the deformation of the pipe when filled with water. So far as the authors are aware, no determination of the magnitude of this moment has been made, and no account of it is ordinarily taken in design and construction.

Steel for Bands.—Bands should be made of mild, open-hearth steel having a tensile strength of from 55,000 to 65,000 lb. per square inch and an elastic limit of not less than one-half the tensile strength. Bands should be capable of being bent cold 180° to a circle equal to the diameter of the band, without sign of fracture.

Threads should be U. S. Standard, and the nuts should fit snugly and yet turn easily without too much play. The bands should be threaded for a length suitable to the diameter of the pipe and sufficient to permit of their easy installation and yet to cinch up the pipe tightly after erection.

Bands are made in one or two pieces for ease of erection. One-piece bands are ordinarily used for pipes up to 52 to 66 in. in diameter. The ordinary practice is to provide a button head on one end and a long thread at the other end, for connection to the malleable iron shoe by means of which the bands are tightened up. This construction permits the button head to lie underneath the threaded end in the shoe, thereby allowing the bands to be spaced as closely together under maximum head conditions as is possible.

A less frequently used construction for moderate heads is made by threading both ends of the band. This necessitates the threaded ends passing one another in the shoe, thereby considerably increasing the minimum spacing between the bands.

With two-piece bands, the usual construction is to provide the lower halves of the bands with threaded ends and the upper halves with button heads.

The threads should be so designed and made that the tensile strength at the root of the threads is equal to that of the body of the bands. Rolling the threads will accomplish this result, and such bands are so guaranteed by the manufacturers. If cut threads are used, the net strength of the band will be that of the area at the root of the thread. Upsetting the ends to a diameter $\frac{1}{8}$ in. greater than that of the body of the rod will give an area at root of thread approximately equal to that of the rod, and permit stressing the rod to its efficient value, with cut threads. If this is done, however, wider shoes will be required to fit the upset ends, resulting in a wider minimum spacing of bands and a lower maximum head under which the pipe may be used. The nuts used are somewhat thicker than standard nuts, so as to avoid the possibility of stripping the threads. Plate-steel washers are used between the nuts and the shoes.

Maximum and Minimum Spacing of Bands.—The maximum spacing between bands is determined by the length of unsupported stave, or of the joint between staves which will not leak under the particular head under which the pipe is used. In present-day practice, this maximum spacing is usually from 6 to 10 and even 12 in. in some extreme cases, and depends upon the thickness of the stave as well as the pressure head. For the thicknesses shown in Fig. 308 and the band spacings as in Fig. 321, the bending moments in the staves between the adjacent bands are well within safe limits. The deflection may, however, be great enough to cause small leaks between staves, if bands of too large cross-sectional area, and consequently widely spaced apart, are used with relatively thin staves. Within the limits of stave thicknesses and sizes and spacing of bands shown in Figs. 308 and Fig. 321, excessive deflections in staves will not occur.

Extra bands are required to support the ends of staves at the joints when the band spacing exceeds 6 in.

The minimum spacing between bands is obtained by staggering the shoes around the circumference of the pipe, usually in groups of three, with one shoe at approximately the horizontal diameter, one above, and one below, as indicated in Fig. 307. This arrangement permits the bodies of two bands on either side to be brought into contact with a shoe, so that half the width of the shoe, plus half the diameter of the band used, determines the minimum spacing and the maximum head under which the pipe may be used.

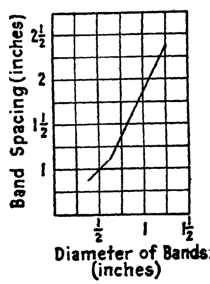

FIG. 310.—Diagram Showing the Minimum Spacing of bands for Wood-stave Pipe Lines.

Figure 310 shows the minimum spacing from center to center of various sizes of bands from $\frac{3}{8}$ to 1 in. in diameter, with shoes of conventional design and for bands having button heads.

It is customary to vary the spacing of the bands for small increments of

pressure head (usually taken at 5 ft. or less), and thus to make the metal in the bands work at practically its full efficiency. In this particular, wood pipe has an advantage over those types of pipe in which the steel shell or reinforcement increases by large percentage increments or must be of a certain minimum thickness for the sake of stability, and hence cannot be as economically employed as the steel used in wood-pipe bands. This advantage is greatest for large pipes at low heads.

For machine-banded pipe, the bands may take either the form of round steel wire, or of flat-steel, galvanized bands, as may be specified or preferred.

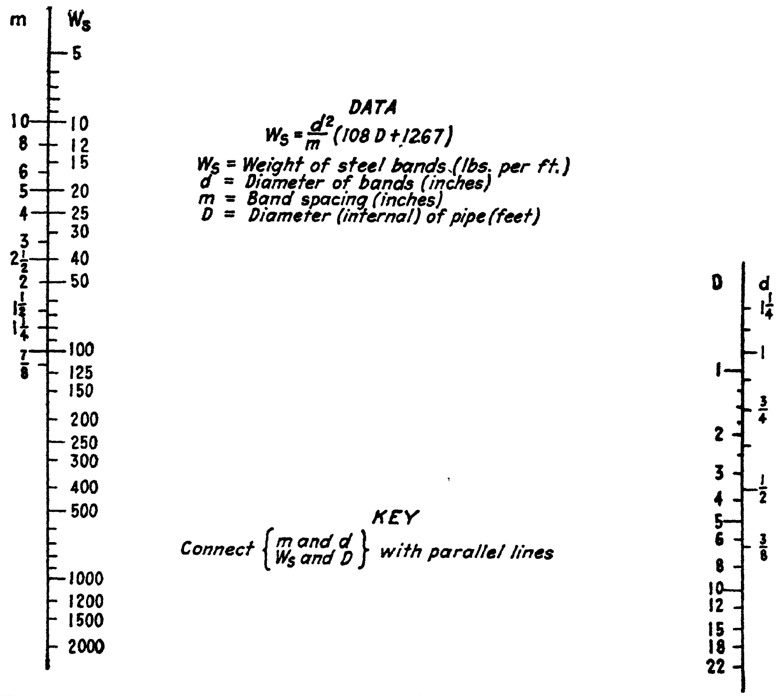

FIG. 311.—Parallel Chart Showing the Approximate Weight of Steel Bands, Shoes and Hardware in Continuous Wood-stave Pipe per Foot of Length of Pipe.

Diameter of Bands.—The maximum cross-sectional diameter of band is largely a matter of convenience in erection. For pipe of, say, 12 ft. diameter, a band $\frac{7}{8}$ in. in diameter is capable of being applied without too much labor in erection. Bands having diameters of 1 in. to $1\frac{1}{4}$ in. can be used if required by the static head, but the labor of erection is of course increased.

The minimum diameter of bands varies with size of pipe, and allowance must be made for the possibility of corrosion and reduction of area, which is, of course, a more serious matter with the smaller than with the larger bands. In practice, $\frac{1}{2}$ in. diameter bands are about as small as should be used for con-

tinuous pipe of 5 ft. or more diameter. Fig. 321 shows the maximum and minimum sizes of bands usually employed. Fig. 311 shows the weight of hardware, including bands, shoes, etc., per linear foot of pipe for different pipe diameters, heads, and sizes of bands.

Bearing on Staves.—Bands should be so designed as not to indent the wood over an area greater than 90° of the circumference of the band. For Douglas fir, the bearing value of bands and their connecting shoes should not exceed 800 lb. per square inch of contact surface; while for redwood, this should be reduced to 600 lb. per square inch.

Such bearing pressures are due to (*a*) the pressure head of water within the pipe, including surges, pressure due to swelling of wood, etc.; and (*b*) the weight of the pipe itself and the contained water.

These two bearing pressures are combined in the case of a pipe supported on one or more bands lying in a cradle. Hence a sufficient number of bands must be provided in each cradle to keep the maximum bearing pressure within the above limits, unless other means, such as grouting between cradle and pipe shell, are resorted to for support, in which case only the pressure due to pressure head will come into play.

In cradles, the pressure due to weight of pipe and water is supported on the horizontal projected length of band lying in the cradle, and is a maximum at the center of the cradle; while the component due to pressure head, etc., is proportional to the equivalent pressure head above any point on the circumference.

235. Tongues or Butt Joints at Ends of Staves.—The ends of adjacent staves are made tight against leakage by means of tongues or butt joints. The simplest form of the tongue type is a rolled steel plate of No. 10 B. W. gage, Fig. 312. These tongues are cut square and true and are usually made $1\frac{3}{4}$ in. wide and of a length $\frac{1}{16}$ to $\frac{1}{8}$ in. greater than the width of the stave at the slot.

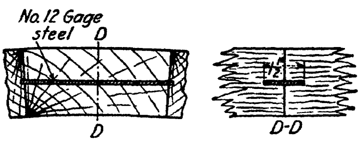

FIG. 312.—Tongue-type Butt Joint.

The slots or saw cuts in the ends of the staves, into which the tongues

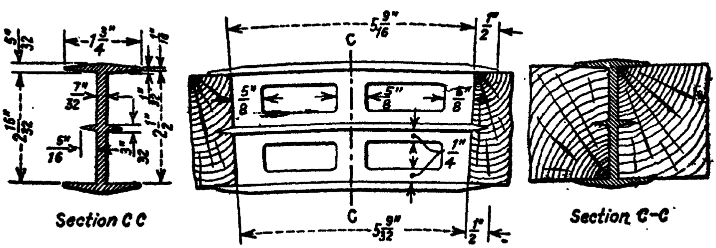

FIG. 313.—Kelsey Malleable Iron Butt Joint.

are inserted, are usually made $\frac{3}{4}$ in. deep, allowing the tongues to cut $\frac{1}{8}$ in. into the wood.

Figure 313 shows the Kelsey butt joint, which makes a stronger joint than

the simple tongue and is particularly applicable to cases where pipe is installed to work under conditions of suction or vacuum, as it transmits the stresses from the joint to all four of the adjacent staves, and aids in keeping the ends of staves from splitting.

In erecting pipe, when entering the tongues or shoes at the end joints, the adjacent staves should be spread apart by means of chisels so as not to score the radial edges of staves unnecessarily.

236. Shoes for Bands.—Shoes should be made of high-class, malleable cast iron, sound, smooth and free from defects.

The shoes that connect the ends of bands should be so designed as to have a strength at least equal to that of the gross cross-sectional area of the bands themselves. Several designs of shoes have been made and used with satisfaction, but by careful attention to design, it has been possible greatly to reduce the weight of metal required in order to meet this specification. Fig. 314 shows a satisfactory type of shoe. In the usual designs, provision has been made for bands having a button head at each end of the top half of the band and threads on the lower half. The most usual design at present used has open-sided slots, which allow the shoe to be slipped under the button head and the other part of the band, with the nut partly screwed on, to be slipped into place without having to slide the shoe endwise over either section of the band. Sufficient bearing area should be provided to keep the bearing on the wood within the limits specified in Sec. 234.

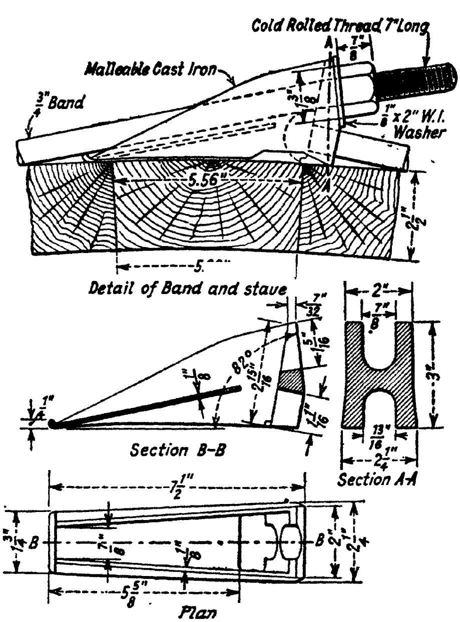

FIG. 314.—Detail of Band Shoe.

Some of the older designs were so made that the shoes had to be slipped longitudinally over the bands; and, in certain of them, the bands were provided with threads and nuts for both top and bottom sections.

237. Allowance for Initial Swelling and Compression of Wood.—In addition to the static head of water at any point, the bands must resist any additional head due to possible surges, and also, at least temporarily, the pressure produced by the initial swelling of the staves when first saturated on filling the pipe. The swelling pressure to be resisted by a single band is that on an an area having a width equal to the radial thickness of the staves and a

length equal to the longitudinal spacing, center to center, of bands. Adams (see Trans. Am. Soc. C. E., Vol. 41, p. 37) concludes that the ultimate pressure due to swelling may be taken safely at 90 to 100 lb. per square inch. In the experiments with both redwood and Oregon fir, quoted by Mr. Adams and in the discussion of his paper by Mr. D. C. Henny, initial swelling pressures up to 300 lb. per square inch were observed, but these soon decreased, as the wood became saturated, to values between 75 and 150 lb. per square inch. Fig. 315 shows the static heads equivalent to a pressure of 90 lb. per square inch for the usual thicknesses of staves and diameters of pipe, which was taken by Mr. Adams as the most probable value as a result of the experiments. The values shown in Fig. 315 are based upon 10 in. spacing between centers of

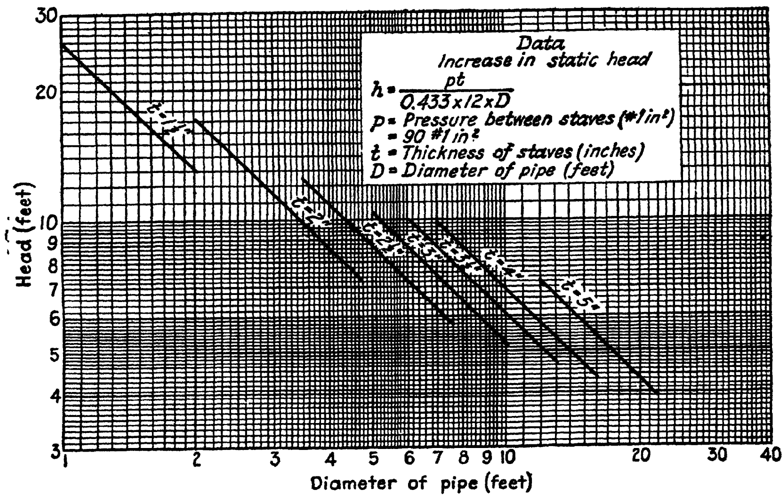

FIG. 315.—Diagram Showing the Equivalent Increase in Static Head on Wood-stave Pipe Caused by Initial Swelling of Wood on Filling Pipe with Water.

bands. After the wood has once become thoroughly saturated and the bands have indented themselves into the staves, this pressure is largely relieved.

From the foregoing, it is evident that the band stress resulting from the swelling of the wood is of greater importance where bands are widely spaced. As the spacing is reduced, this stress becomes a smaller portion of the total stress which the bands must resist. Since the initial swelling pressure decreases rapidly, it is not necessary to take the excess over the ultimate pressure into account, further than to keep the initial band stress below the elastic limit of the steel. Some of the wood-pipe manufacturers disregard the swelling pressure entirely in designing, on the assumption that it practically disappears within a few weeks after the pipe is put in service.

When the initial stress due to swelling is high, it is advisable to fill the pipe slowly, thereby affording an opportunity for the swelling pressure to decrease before subjecting the pipe to the full static head.

238. Minimum Allowed Radius.—Continuous wood pipe has a certain degree of flexibility and may be laid to any desired radius, not less than approximately fifty times the normal diameter of the pipe. Somewhat sharper curves can be used but will cause difficulty in erection. Horizontal curves are formed by pulling the pipe into line by means of chain blocks after a few bands are in place. This becomes very difficult if too small radii are attempted.

239. Pipe Supports.—Some form of exterior support or saddle, extending at least partially up the sides of continuous wood-stave pipe, is usually necessary. These supports range in form from (1) a simple cross sill of timber, with additional blocks of wood on either side, which are cut to the radius of the outer circumference of the band; to (2) a concrete, masonry, cast-iron or structural steel saddle extending up toward or above the horizontal diameter of the pipe, formed to the external radius of the bands, or, at least, supporting the pipe at three or more points. Typical saddles, which are also suitable for steel pipe, are given in Figs. 293 to 297, inclusive.

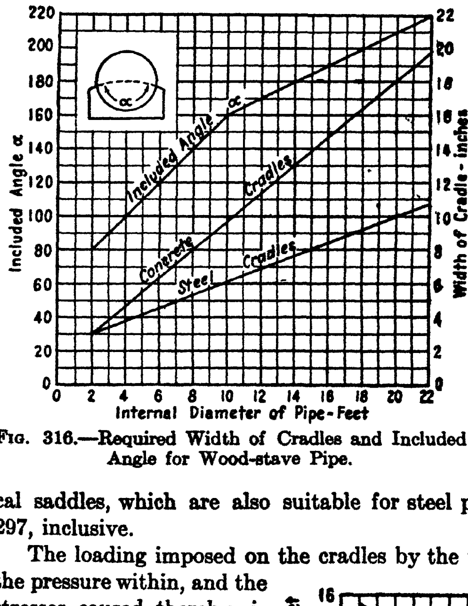

Fig. 316.—Required Width of Cradles and Included Angle for Wood-stave Pipe.

The loading imposed on the cradles by the weight of the pipe itself, plus the pressure within, and the stresses caused thereby, is determined in the same manner as for steel pipe. The reader is referred to Sec. 221 for a diagram showing these loadings for steel pipe. This diagram is also applicable to wood pipe.

The current practice as to spacing and width of cradles longitudinally of the pipe line is shown in Figs. 316 and 317 in which the maximum and recommended values are graphically shown. The area of contact between the cradle and the pipe should not be smaller than that required to give a bearing

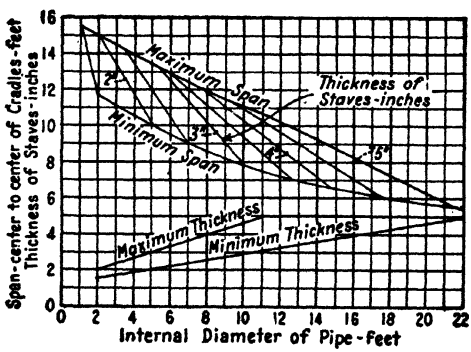

Fig. 317.—Spans for Wood-stave Pipe.

pressure between the bands and the wood of 800 lb. per square inch for fir and pine, and 600 lb. per square inch for redwood. Carrying the cradle up to the horizontal diameter of the pipe greatly increases its ability to withstand external loading, crushing or vacuum.

As the diameter of a pipe increases, the tendency of the upper half to collapse or bulge out over the tops of cradles also increases. For this reason the larger the diameter of the pipe, the larger should be the portion of the circumference supported in the cradles. In Fig. 316 the line marked "Included Angle α" indicates the angular portion of the pipe circumference which should be supported.

From the above, it is apparent that as the diameter of pipe increases, so also should the thickness of the cradle and the angle of contact.

Where the spacing between cradles is too great, the pipe flattens at the cradles and sags between them.

Particular attention should be given to foundation conditions, to insure that the saddles will not settle or get too far out of line. In this particular, wood pipe possesses an advantage over other types of pipe in that it is flexible and can accommodate itself to slight changes of grade and alinement without being seriously damaged or having its usefulness impaired.

240. Painting and Creosoting.—The most common method of treatment is by the pressure-vacuum creosoting process, which leaves a residue of about 8 lb. of creosote oil per cubic foot of wood. This treatment will be effective over varying periods of time, depending upon the climatic conditions, the character of the wood, and whether or not the pipe is exposed or buried. If the pipe is laid in the open, such treatment can be renewed by a brush or air-sprayed coat or coats of light creosote oil as conditions may require.

Staves tend to decay at the ends and where they are bruised. It is therefore desirable to dip the ends in creosote oil or other preservative before laying in the pipe. Creosote oils appear to be the most suitable for the purpose as they are not soluble in water and inhibit rot and fungi.

Experiments made with Douglas fir creosoted by the high-temperature process show a loss of strength up to 30 per cent. Therefore, the low-temperature process should be used for staves.

A method which appears to give satisfactory results is that of laying the pipe of untreated staves, then spraying or painting it with creosote oil before it is filled with water, and, at a later time, after the more volatile oils have evaporated, painting it with a tar- or asphalt-base paint. Some question appears to exist as to whether or not there is an interaction between creosote oil and asphalt-base paints. Either tar or asphalt paints will serve as efficient protection to the steel and metal parts of the pipe.

This method of treatment was applied with satisfactory results to a pipe line while in operation. It was, of course, necessary to stop all leaks carefully in order to insure penetration of the outer surface of the pipe by the creosote.

Ordinary paints have been applied to wood pipe, but the objection appears to be that they require more frequent renewal in order to preserve the pipe satisfactorily.

When repairs or replacements become necessary, it is sometimes found

that, if not protected, the threads on the bands have become so rusted that it is impossible to get the nuts off. It is then necessary to cut the bands, and the loss of the bands and nuts results. Initial painting and occasional repainting of threads with asphalt- or tar-base paint will obviate this difficulty.

241. Life of Wood-stave Pipe.—The life of wood-stave pipe has been the subject of much discussion and, in general, it may be said to depend upon:

(1) Its design and the method of installation, whether exposed, half buried, or buried.
(2) The nature of the soil with which it comes in contact.
(3) The protection afforded to the staves and metal appurtenances, i.e., whether treated or untreated, with paint or other preservative, galvanized, etc.
(4) Maintenance after installation.
(5) Whether the pipe is continuously filled with water or intermittently filled and emptied and allowed to dry out.
(6) Climatic conditions.
(7) The wood of which it is composed.

The maximum life of wood-stave pipe, under favorable conditions, is not definitely known; but there are many examples of pipes made of what are now considered inferior materials, such as hemlock, for instance, which have been in more or less satisfactory service for periods of forty to fifty years; and, as a rule, the timber and metal in these pipes were not treated or painted. Other examples of pipes that have given very short service are numerous.

If a careful and intelligent study and appraisal of the conditions affecting the life of a given installation is made, suitable designs, materials and protection provided, and the pipe and related structures properly inspected and maintained, a useful life of from thirty to fifty years should be attained for a pipe continuously filled with water, except under unfavorable conditions which will be more fully discussed in the paragraphs following.

In general, it may be stated that a wood-stave pipe should always receive preservative treatment unless a life of from five to twenty years, depending on the surrounding conditions, is all that is required. As is elsewhere pointed out, it is a great advantage to have the entire pipe exposed so that it may be readily inspected and the preservative treatment renewed when necessary. If this is done, the maximum useful life may be attained.

(1) *Design and Installation.*—When the use to which the pipe is to be put is temporary or for a relatively short period of time, the question of the manner of installation is not very material. If, however, long life is desirable or essential, experience indicates that the maintenance of pipe in good condition is much facilitated by installing it entirely in the open on suitable cradles. A pipe so installed can be easily and frequently inspected so that any repairs or maintenance can be quickly and easily accomplished before serious injury or damage results.

A pipe laid underground has a tendency to decay under the following conditions:

(*a*) If the soil is particularly porous so that the wood is not continuously wet but is exposed to alternate wetting and drying;

(*b*) If vegetable mold and certain fungi are contained in the backfill material, a condition which has resulted in very rapid deterioration of some large pipe lines;

(*c*) After a long period of saturation, the staves become very pliable and, where small leaks have washed away the loose material beneath the pipe, settlement may take place at the sills and over other solid objects. This tends to deform the pipe and force individual staves inward or break them, causing serious leaks.

A pipe line that is partially buried has a tendency to decay near and at the ground line, due to the alternate wetting and drying of the staves. The portion underground is, of course, subject to the same influences as a buried pipe.

Sufficient space should be left under and around a pipe laid above ground to insure, so far as possible, that no earth or other material can find its way into the trench and remain in contact with the pipe; or, at least, means should be provided for removing such material soon after it is deposited.

For the larger sizes of pipe, some form of support or cradle at frequent intervals is very desirable and, in fact, practically essential. No positive formula has been worked out, but in practice it is customary to space such supports as described in Sec. 239, " Pipe Supports." There is an instance on record of a large wood pipe remaining intact and filled with water after the support had been washed out from under a section 50 ft. long. Had the pipe been emptied and allowed to dry out, this section would undoubtedly have collapsed.

Wood pipe 6 ft. or more in diameter and of normal design, made of staves of the thicknesses above mentioned, should not be backfilled to a depth of more than 2 to 4 ft. over the top and should not be subjected to heavy loads.

Smaller pipes have successfully stood up under as much as 10 ft. to 12 ft. of backfill. In those cases, the fill was of good material, firmly tamped and not subjected to washing from leaks, etc. Great care should be exercised in the installation of pipes under heavy fills. Where roads or heavy loads over such pipes have to be carried, bridges or other supports, external to the pipe, should be provided. If heavy backfill is to be placed over a large pipe, thicker staves, and possibly a concrete jacket around the pipe, should be provided.

Wood pipe for use where suction or vacuum may be produced, as in pump suction or condenser lines, should be designed with thicker staves, and some device, such as the Kelsey butt joint, may be used between ends of staves to distribute the stresses. If the pipe is large and the vacuum high, the safety of the pipe may require that it be encased in concrete.

The design and installation of a satisfactory wood-stave pipe involves special knowledge, only obtainable from actual experience. It will usually be found advisable to entrust the construction to one of the established manufacturers and to consult with their engineers in regard to design and layout, thus taking advantage of their experience, which is bound to be much more extensive than that of most designing and construction engineers.

In backfilling around wood-stave pipes, care should be taken to compact the fill as it progresses up the sides of the pipe. Backfill should not be placed to too great a height on one side of the pipe, or it may be forced out of line.

This point should be particularly borne in mind if the backfill material is hydraulically sluiced in or kept saturated with water. In one case, neglect of this precaution in the covering of a pipe under pressure resulted in opening of the end joints and abnormal leakage. After the pressure was removed, the semi-fluid material crushed a considerable length of pipe.

A case is also on record of a wood pipe which was crushed in by the weight of 18 in. of earth on the top, when the water was drawn off after the fill had been removed from one side.

(2) *Nature of Soil.*—In alkali soils, serious or complete rusting of bands and other metal appurtenances of pipe lines has frequently been reported. In such soils, it is probable that long life can be expected only from pipe laid entirely above ground.

In one case, early and serious decay of the wood of an untreated pipe was traced to the presence of fungi in the backfill material, regardless of the head on the pipe. Roots, grass, etc., should therefore be excluded and only sterile earth used for backfilling an untreated wood pipe.

(3) *Protection: Painting and Treatment of Staves.*—Many pipes have been laid without any treatment of the staves. The useful life of wood depends upon its ability to resist decay and the attacks of fungi. Hence, treatment with a preservative, which inhibits such attacks, and renewal of the treatment at suitable intervals, will manifestly lengthen its life. Such treatment is described in Sec. 240.

(4) *Maintenance.*—A pipe should be inspected at sufficiently short intervals to insure that no serious deterioration has taken place since the previous inspection. Any serious leaks should be stopped up by the means suggested in Sections 232 and 246 and renewal of the protective coating or treatment should take place whenever inspection reveals that such is necessary. Also all structures and appurtenances of the pipe line should be renewed or repaired whenever any sign of failure or damage is noticed. If these precautions are taken, the life of any pipe line is bound to be greatly extended.

(5) *Pipe Continuously or Intermittently Filled.*—A pipe line that is continuously filled with water will not suffer as much deterioration as one that is emptied and allowed to dry out for longer or shorter intervals of time. Preservative treatment is a necessity if reasonable life is desired.

(6) *Climatic Conditions.*—In extremely hot, dry climates, as in the arid regions of the western United States, excessive checking of the staves occurs. In climates where hot, dry periods alternate with wet periods, allowing the exterior of the wood to become alternately saturated and dry, decay is hastened. Preservative treatment will greatly minimize both of these conditions and prolong the useful life of the pipe.

(7) *Wood for the Staves.*—The kind of timber used for the staves has a marked effect on the life of the pipe, as explained in Sec. 233.

A report of a recent investigation by the U. S. Reclamation Service, quoted from an article by Mr. W. H. Nalder, furnished by the Denver office of the Service, has resulted in the following conclusions as to conditions tending to shorten the useful life of wood pipes. The results of this investigation confirm many of the preceding statements:

(1) Intermittent service. Pipes full of water during the irrigation season and emptied during the next irrigation season.
(2) Installations made in gravelly or open soils.
(3) Installations made in alkali soil, resulting in destruction of the joints and wire winding.
(4) Installations of machine-banded pipe in which separate wooden collars are used at joints and in which these collars tend to rot out while the pipe walls remain in good condition, due evidently to the fact that the collars are not as thoroughly saturated in service as the walls of the pipe.
(5) Installations under low hydrostatic head, say, less than 15 ft., which is not sufficient properly to saturate uncreosoted wood.

242. Expansion Joints not Necessary.—Owing to the low coefficient of expansion of wood, the relatively short lengths of staves, and the fact that the end joints are made tight by tongues or butt joints, which permit a slight movement without causing perceptible leakage, expansion joints in wood pipe are unnecessary except at connections to other structures or other types of pipe.

243. Freezing.—No record of serious effects in wood-stave pipe of the sizes and lengths used in hydro-electric installations is available. A very recent account of freezing in a 26½-mile section of municipal water supply pipe in the State of Washington is given in *Engineering News-Record* of December 31, 1925, page 1077. Eight miles of this approximately 28-in. diameter line was left exposed, resulting in the formation of some ice each winter. In the winter of 1924 and 1925, a 10-in. thickness of ice formed, reducing the interior diameter of pipe to 8 in. This ice was subsequently removed at a sand box in the line when the weather became warm enough to permit the ice to thaw.

In this particular case, it was found that freezing caused spalling or breaking off of the outer half of the ends of the staves over the steel-plate butt joints. This appears to be an exceptional case as a similar condition has not been reported elsewhere. These breaks were repaired by placing galvanized iron plates over each break and fastening on by means of two additional bands, quite similar to the repair mentioned in Sec. 246.

In freezing climates, snow invariably collects to greater or less depths on the top of wood pipe, and undoubtedly helps greatly to insulate the pipe against freezing. No cases have come to the attention of the writer where any serious loss of head has been observed in wood pipe during freezing weather. It is therefore probable that very little ice forms inside such pipes, except perhaps where a pipe is laid up and no water permitted to flow through it for a considerable length of time.

244. Accessories.—*Connections to Steel Pipe.*—Wood pipe should be connected to steel pipe by means of a slip joint, on account of the possibility of expansion and contraction of the steel pipe. Fig. 318 shows a typical slip-joint connection which has been satisfactorily used. A joint should contain a retaining ring at the inner end, in front of which oakum or hemp packing is tightly driven up and, in the simplest possible case, retained by a retaining ring as indicated.

Under heavy pressures, such a joint will leak more or less. If leakage is

objectionable, it will be necessary to install a ring of Z-section, which is capable of some movement and can be tightened up by means of nuts on the studs shown in the figure. Where a joint made as above described cannot be satisfactorily kept tight, lead wool may be tightly calked in for the last 2 in. or more at the outer end of the joint, after which the retaining ring is to be put on and tightened up. Another method is to pack the joint tightly with oakum at the inner end, then pack tightly with dry cement mortar to within 3 in. from the outer end, and the remainder with lead wool tightly calked, and then tighten up the retaining ring.

The annular space outside of the wood varies from 1 to 3 in. in width in different designs. The smaller the space in which the oakum or other packing can be tightly calked, the better are the results obtained.

At such joints, the steel pipe should, if possible, be securely anchored in a

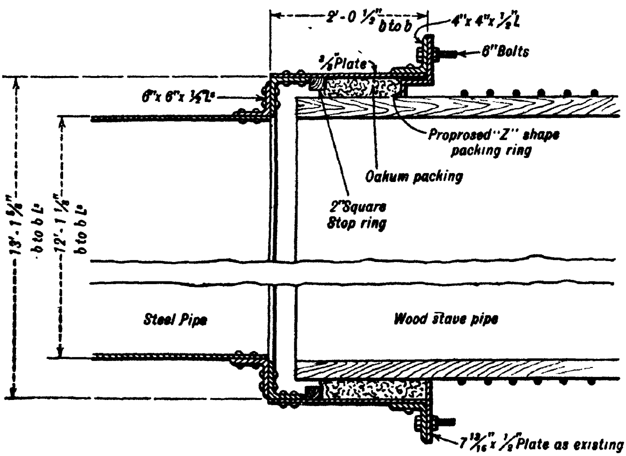

Fig. 318.—Detail of Trenton Falls Expansion Joint at Junction of Steel and Wood-stave Pipe Lines. Utica Gas and Electric Co.

concrete block, as otherwise appreciable movement and leakage may take place.

Connections to Concrete Pipe, Dams, etc.—For connecting wood pipe to concrete pipe or other concrete work, as at dams, etc., an annular joint, quite similar to that just described for steel pipe, should be provided. This joint will not differ essentially from that described above, except that where connection is made into a large mass of concrete, the retaining ring may not be required, particularly under low heads. If the retaining ring is omitted, the outer portion of the packing should preferably be of lead wood solidly calked in.

Connections for Surge Tanks, Vents, Man-holes, Drains, etc.—A typical vent-pipe connection is shown in Fig. 319. Connections for man-holes, drains, or other openings of less diameter than the normal diameter of the pipe are

most readily made by means of castings which are in general of the form suggested in Figs. 319 and 320.

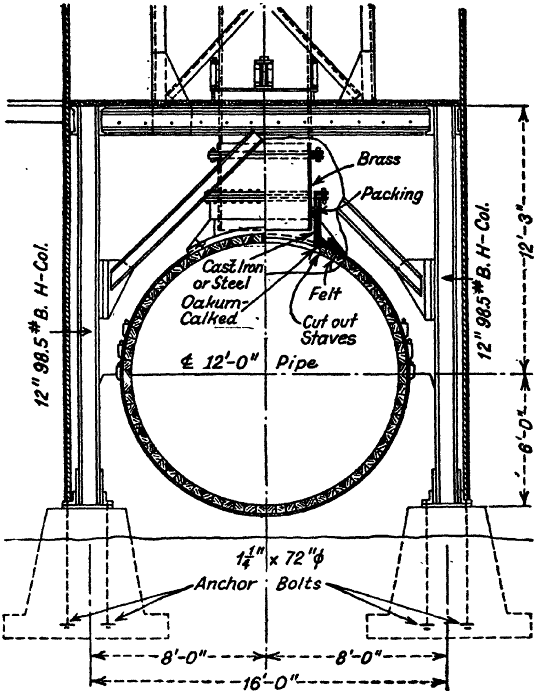

FIG. 319.—Typical Connection of Steel Vent Pipe.

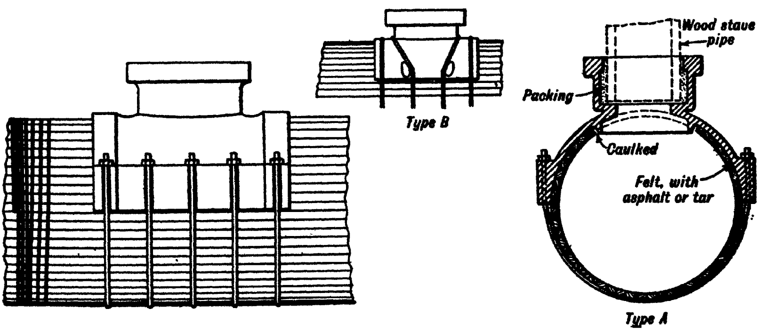

FIG. 320.—Typical Branch Connection for Wood-stave Pipe.

The essential features of such connections are a metal pad, usually of cast iron, but preferably of steel for high pressures, fitted to the outer circumfer-

ence of the staves and provided with lugs corresponding to the shoes used for connecting the bands. In addition, a hole, which closely fits a lip formed on

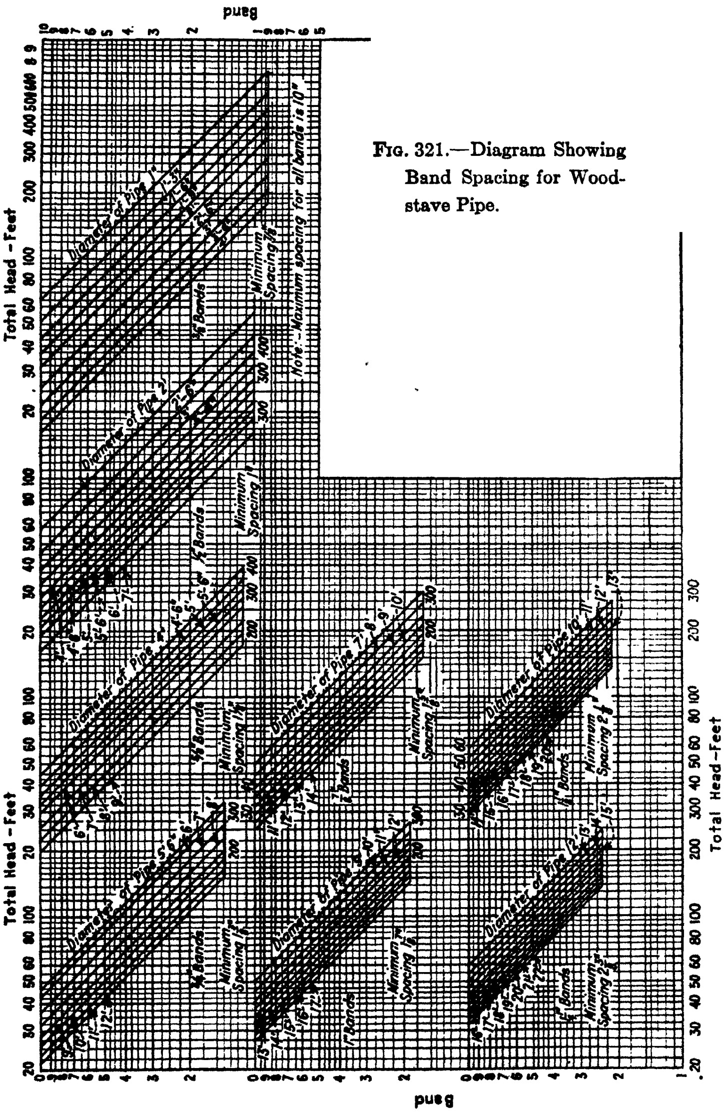

Fig. 321.—Diagram Showing Band Spacing for Wood-stave Pipe.

the inner side of the casting, should be sawed in the staves. Water-tightness may be secured by means of sheets of felt saturated with asphalt and laid between the casting and the staves. Calking, between the projecting lip and

the wood, with oakum saturated with asphalt, also aids in making such a joint water-tight.

Sections of bands are cut off and threaded to suit the lugs on the sides of the casting, and are made of such lengths as to secure the proper staggering of the shoes at the sides of the pipe. For tall and heavy vent pipes, etc., it is necessary so to arrange the design, as indicated in the cut, Fig. 319, that the vent pipe itself is supported exterior to the pipe and the slip joint provided with a packing gland, so that the "breathing" of the pipe can take place independently of, and without causing movement of, the vent pipe or surge tank. The inner member of the slip joint should preferably be made of brass so as to have no tendency to rust fast to the iron or steel casting attached to the pipe.

Special Bends.—Where the radius of curvature is smaller than that to which the wood pipe can be successfully bent, it is necessary to install special bends which may be made of plate steel, cast iron or concrete. These bends should be provided with joints as described above, under "Connections to Dams, etc." and should be securely anchored against movement.

245. Special Requirements and Precautions.—In laying out wood pipe, it is desirable to provide sufficient space between adjacent pipes and between pipes and other structures, such as retaining walls, piers, etc., to permit ratchet wrenches to be operated in tightening the nuts or cinching the pipe, and also to permit workmen to go completely around the pipe, for the purpose of erection, inspection, etc. The minimum space for this purpose is about 18 in.

In laying out the pipe, it is desirable to keep the profile below the lowest possible hydraulic gradient under extreme operating conditions. If any points fall above such gradient, adequate vents, air valves, or vacuum valves should be provided. Such vents and valves should be completely protected against freezing, as otherwise they might be inoperative when required to function, and in several instances, their freezing has resulted in collapse.

Every precaution should be taken to insure the safety of a pipe line against damage or failure due to washouts from any cause. Wherever possible with economy, foundations for supporting structures should always be carried to rock or other firm bearing strata. Where streams flow alongside or underneath a pipe, suitable retaining walls or other construction, to limit the possibility of washout in cases of flood, should be provided.

In sandy or easily eroded soils, a very satisfactory and efficient precaution against serious scour was provided in at least one case by driving rows of short sheet piling at right angles to the center line of the trench and carrying them a short distance up the sides of the trench, so that any conceivable volume of water from ordinary leakage would be compelled to flow over the top of the successive rows of sheeting. These rows were spaced sufficiently close together, so that even on rather steep grades there was but little vertical fall between successive rows. Where the fall was considerable, the length of sheeting was made at least twice the vertical height between successive rows.

On fills, unless they are of extremely porous material, not easily eroded, it is important that means be provided for controlling the flow of water resulting from leaks, and for carrying it to and down paved or otherwise protected

channels to a safe distance from the pipe line. Unless this is done, there is a possibility of a washout due to leaks, which occur when the pipe line is first filled, and may also occur at some subsequent time.

A pipe should also be protected from the possibility of being floated by the presence of water around it at a time when it is empty. Likewise, sufficient drainage should be provided in the case of a pipe line buried in soft or unstable soil, so that the backfill cannot become completely saturated and crush the pipe in case the water is drawn out of it.

When it becomes necessary partly to uncover a section of buried pipe more than a few feet in length, it is important to uncover it completely, down nearly or quite to the horizontal center line, so as to relieve any over-burden on top of the pipe in case the water is drawn off. Where this has been neglected, serious collapse has taken place.

246. Maintenance.—The maintenance of buried or partly buried pipe is complicated by the fact that it is difficult, if not impossible, to locate leaks. Cases have occurred where leaks have only been found after excavating for considerable distances along the pipe.

One occasionally finds leaks that cannot be closed by the customary plugs and wedges. In several cases, steel plates of approximately No. 10 gage, rolled to the radius of the outer circumference of the pipe, were inserted under the partly loosened bands, after which the bands were again tightened, with satisfactory results.

A repair was made to a large pipe, in which the staves were deeply rotted, by cutting away the decayed portions, wedging under the bands with pieces of wood, and applying 2 : 1 Portland cement mortar reinforced with No. 10 iron wire of the sort that is ordinarily used for tying concrete forms.

247. Specifications for Continuous Wood-stave Pipe.—*Staves.*—All lumber used in staves shall be of untreated Douglas fir or redwood or creosoted Douglas fir, as provided in the schedule. All lumber in staves shall be sound, straight-grained, and free from sap, dry rot, checks, wind shakes, wane and other imperfections that may impair its strength or durability. Pitch seams will be permitted in not over 10 per cent of the total number of pieces, if showing on the edge only, and if not longer than 4 in. nor wider than $\frac{1}{16}$ in. No through knots nor knots at edge nor within 6 in. of end of staves will be allowed. Sound knots not exceeding $\frac{1}{2}$ in. in diameter, appearing on one face only and not falling within the above limitations, will be accepted. All lumber used shall be thoroughly seasoned. All staves shall have inside and outside faces accurately milled to the required circular arcs to fit a standard pattern provided by the contractor. Staves shall be trimmed perfectly square at ends and the slots for tongues shall be in exactly the same relative positions for all ends. Staves shall have an average length of not less than 14 ft. and no staves shorter than 8 ft. will be accepted. The thickness of staves shall be not less than (   ) inches. All staves shall be of uniform cross-section. Staves to be creosoted shall conform to the above specifications for untreated staves except that bright sap will be allowed on the inside of the staves in not over 10 per cent of the total number of pieces to a maximum extent of one-fourth the thickness of the stave. (Many engineers permit sap to this extent in untreated fir, where pipe will be continuously filled with water.)

Bands.—A pipe band shall consist of one complete fastening and shall include the bolt, shoe, nut, and washer necessary to form the same. Bolts used in bands shall be made in one or two pieces which shall completely encircle

the pipe. The distance center to center, of bands shall be as shown on the profile, except that the spacing of bands at butt joints shall in no case be greater than 6 in.

Bolts.—All bolts shall be of steel of the diameter stated in the schedule. Bolts may have either button or bolt heads provided that the strength of the head shall be not less than that of the bolt. The bolts shall be as strong in thread as in body, and threads shall permit the nut to run freely the entire length of the thread. The threaded length shall be ____ in. and there shall be ____ in. of available thread, measured from face of shoe, remaining after final cinching. Bolts shall be formed to shape ready for installation and shall be bundled into suitable packages for protection and convenience in shipping and handling. The material for bolts shall conform in all respects to the Standard Specifications for Structural Steel for Buildings, as given in the latest volume of the "A. S. T. M. Standards," published by the American Society for Testing Materials. Nuts shall be of such thickness as to insure against stripping of threads and shall be furnished 2 per cent in excess of the net number required.

Shoes.—The shoes shall be of malleable iron, shall fit accurately the outer surface of the pipe, and shall have sufficient bearing surface at the head to prevent injurious indentation of the wood. Bidders are required to submit for approval a drawing or sample of the shoe they propose to furnish. If required, such shoes shall be shown under suitable tests to be stronger than the bolt. Malleable iron for shoes shall conform to the "Standard Specifications for Malleable Castings," as given in the latest volume of the "A. S. T. M. Standards," published by the American Society of Testing Materials.

Tongues.—Metal tongues of 12 gage galvanized steel or iron or other non-corrosive metal acceptable to the engineer shall be furnished for joining the ends of the staves. These tongues shall be 1¼ in. wide and of a length, such that when in place they will penetrate into the sides of the adjacent staves without undue injury. The tongues and slots shall be so proportioned as to insure a tight fit of tongues into slots without danger of splitting the staves. The tongues shall be of true rectangular shape, straight and smooth and free from ragged edges. The number of tongues to be furnished shall be not less than 2 per cent in excess of the number of sticks of staves furnished and they shall be suitably bundled or boxed for shipment.

In cases where vacuum or the pressure of excessive backfill must be resisted, special galvanized malleable iron butt joints of a type satisfactory to the engineer may be specified in place of the sheet-metal tongues.

Coating of Bands.—All bolts, shoes, nuts and washers shall be coated by being dipped when hot in pure California asphalt, or its equivalent, or other suitable protective paint or covering, applied cold, preferably by dipping. Bolts shall be bent to the required arc, and shall be provided with nuts, before dipping. If the material is dipped cold, it shall be left in the hot bath a sufficient length of time to insure that it has acquired the temperature of the asphalt before removal. The coating shall be so proportioned and applied that it will form a thick and tough coating, free from tendency to flow or become brittle under ordinary ranges of temperature.

Creosoting.—The creosote oil used and the method of treatment shall conform to the latest standard specifications of the American Wood preservers' Association for Creosote Treatment of Douglas Fir Wood-pipe Staves. All timber to be creosoted shall be thoroughly kiln- or air-dried and milled before treatment. Treatment shall be by the hot (see Sec. 240) and vacuum process and a sufficient quantity of oil shall be forced into the wood to assure the retention of at least eight (8) lb. of creosote oil per cubic foot of timber after the final vacuum has been drawn. The entire process shall be carried on in such a manner as to guarantee the treatment specified without warping or otherwise damaging the timbers, and to leave them free from excess surface oil.

Erection of Pipe.—The pipe shall be built in a workmanlike manner. The ends of adjacent staves shall break joints at least 3 ft. The staves shall be driven in such a manner as to avoid any tendency to cause wind, and the required grade and alignment must be maintained. Staves shall be driven to produce tight butt joints, driving bars or other suitable means being used to avoid marring or damaging staves in driving. In rounding out the barrels care should be taken to avoid damage by chisels, mauls or other tools. The barrels shall be rounded out to produce smooth inner and outer surfaces and no diameter at any point shall deviate more than 1½ per cent from the average diameter at that point. Rods and bends shall be accurately spaced as shown on the drawing and placed perpendicular to the axis of the barrel, except that the spacing of bands at butt joints shall in no case be greater than 6 in. Shoes shall be placed so as to cover longitudinal joints between staves and bear equally on two staves as nearly as practicable. They shall be placed alternately on opposite sides of the pipe if single-piece bands, or staggered in groups of three if two-piece bands. Shoes shall not be allowed to cover the butt joints. Bolts shall be hammered thoroughly into the wood to secure a bearing on 60 degrees of the circumference of the bolt. All kinks in bolts and rods shall be carefully hammered out. Bands and rods shall be back-cinched to the satisfaction of the engineer so as to produce the required initial compression stress in the staves. After erection the contractor shall retouch all metal work where abraded with an asphaltum or other paint satisfactory to the engineer. All metal work shall be handled with reasonable care so as to avoid injury to the coating as much as possible. In hammering shoes into place they shall be struck so as to avoid deformation or injury. If staves treated with a preservative are furnished, they shall not be cut or sawed except when such cutting or sawing is unavoidable. All such cut ends and untreated surfaces shall be given one coat of wood preservative by the contractor. The ends of the staves at either end of the pipe shall be sawed off true and at right angles to the longitudinal center lines of the pipe and connection made into the adjacent pipe thimbles or structures and the joints made tight by packing with oakum and asphaltic composition or otherwise, as directed by the engineer.

Test for Water-tightness.—Upon completion of erection the contractor shall test the pipe for water-tightness under full hydrostatic pressure. All leaks found at the time of making the test shall be repaired by the contractor. If the leakage at any point is not so large as to endanger the foundation, the pipe shall be kept under full pressure for two days before repairing is started, in order to allow the wood to become thoroughly saturated.

N.B. The above and also the following specification follow closely those of the United States Reclamation Service except when alternatives (in parenthesis) show what is considered good commercial practise where pipes are in continuous use and filled with water, as in hydro-electric plants.

248. Specifications for Machine-banded Wood-stave Pipe.—*Staves and Couplings.*—Specifications are the same as given above for continuous wood-stave pipe, with the following exceptions:

(*a*) Edges of staves are to be milled radial with a small tongue and groove and omitting the slots for tongues at the ends of staves.

(*b*) Machine-banded pipe to be made up in random lengths averaging about 12 ft. and a minimum length of 8 ft. (The United States Reclamation Service specifies an average length of not less than 15 ft. and minimum length of 8 ft.) These dimensions may be increased or diminished accordingly as ease of transportation, which would be facilitated by short sections; or reduction of leakage, which would be facilitated by long sections, is the important point in the installation.

Diameter of Pipe.—No diameter of any pipe shall differ more than 1½

per cent from the specified diameter, and the average of the vertical and horizontal diameters at any point shall not be less than the specified diameter.

Thickness of Staves.—Unless otherwise stated in the schedule, the finished thickness of staves shall be as follows:

4 to 6 in.	$1\frac{1}{16}$ in.
8 to 10 in.	$1\frac{1}{8}$ in.
12 to 14 in.	$1\frac{3}{16}$ in.
16 to 18 in.	$1\frac{1}{4}$ in.
20 to 24 in.	$1\frac{5}{16}$ in.
26 to 30 in.	$1\frac{7}{16}$ in.
32 to 36 in.	$1\frac{1}{2}$ in.

Lumber for Staves.—Specifications same as for continuous pipe.

Banding.—Size and spacing of banding wire shall be adjusted for a working stress of 12,000 lb. per square inch on the wire for the maximum head to which the pipe is to be subjected. (This is very conservative, 15,000 lb. per square inch is usually specified.) The wire spacing shall in no case be greater than shown in the following table, nor greater than will produce a pressure of wire on the wood of 800 lb. per square inch as calculated from the formula:

$$B = 0.868\frac{SHR}{d(R + T)},$$

where B = pressure on wood in pounds per square inch;
H = head in feet on center of pipe;
S = spacing of wire in inches;
R = inside radius of pipe in inches;
d = diameter of wire in inches; and
T = thickness of staves in inches.

The minimum gage of banding wire and the maximum wire spacing to be used on pipes of various diameters shall be as follows:

Pipe Diameter Inside, (Inches)	Gage (Am. Steel and Wire Co.)	Max. Allowable Spacing, C. to C. of Wire (Inches)
2 to 4	No. 8	2
6 to 8	No. 6	3
10 to 14	No. 4	3
16 to 20	No. 2	$3\frac{1}{2}$
22 to 24	No. 1	$3\frac{1}{2}$
26 to 30	No. 0	4
32 to 36	No. 00	4

Wire shall be of medium steel, galvanized, and shall have an ultimate tensile strength of 55,000 to 65,000 lb. per square inch, an elastic limit of at least one-half the ultimate tensile strength, and capability of being bent cold on itself without fracture. The bidder shall state in the schedules the size and spacing of banding wire he proposes to furnish.

Individual Bands.—Unless otherwise noted in the schedules, individual bands shall be used for reinforcing joints, as specified in paragraph below. The smallest bolts used shall be $\frac{3}{8}$ in. in diameter. The bolt shall have an ultimate tensile strength of 55,000 to 65,000 lb. per square inch; an elastic limit of one-half the ultimate tensile strength; and capability of being bent cold without fracture 180 degrees around a pin having a diameter equal to the

thickness of the test piece. Individual bands shall have cold-rolled threads or cut threads on upset ends and shall be provided with the necessary shoes, nuts, and washers. The shoes shall be malleable iron and shall be stronger than the bolts, with sufficient bearing on the wood at the tail to prevent injurious indentation in cinching. The shoes shall be true to pattern and free from blemishes, scale and shrinkage cracks, and shall have a workmanlike finish. Bidders shall submit samples or drawings of the type of shoe they propose to furnish.

Joints.—Joints shall be of the type specified in the schedule. Unless otherwise noted in the schedule, all joints shall be reinforced with individual bands designed to make the banding at the joint 50 per cent stronger than the banding on the pipe.

Coating.—After manufacture the outside of the pipe shall be dipped in a bath of hot coal tar and asphaltum. The coating material shall adhere strongly to the pipe and banding wire and shall not flow or become brittle under the ordinary range of temperatures. Care should be exercised to keep the coal tar and asphaltum from the tenon ends and inside surfaces, and if necessary, the tenons shall be wrapped with paper while being dipped. After dipping, the pipe and collars shall be rolled in fine sawdust while the coating is still soft.

Inspection.—Inspection of pipe will be made at the mill, but the contractor will be held responsible for any damage in transit caused by improper loading of the pipe.

249. Bibliography.—

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3. Modern Practise in Wood-stave Pipe Design and Suggestions for Standard Specifications, by J. F. Partridge. Trans. Am. Soc. C. E., Sept., 1918, Vol. LXXXII, pp. 433–514.
4. Wood-stave Pipe on Projects of the Reclamation Service, by W. H. Nalder. Reclamation Rec., Vol. 12, pp. 567–9, also Eng. News, Vol. 88, pp. 493–4, 1921. Concisely tabulated data on 196 installations of wood-stave pipe.
5. Reconditioning Wood-stave Pipe Line, by J. B. Holdcroft, Canadian Engineer, Vol. 46, pp. 447–9, 1924. Repairs made on 72-in. continuous pipe line at Anyox, B. C., which had deteriorated on account of severe operating conditions. Gives suggestions for preserving wood pipe and care of bands and shoes.
6. Wood-stave Pipe Lines and Penstocks, by J. B. Holdcroft. Canadian Engineer, Vol. 47, pp. 149–51, 1924. Design of wood-stave pipe employed for hydraulic power installations; dimensions of staves, design of tongues, bands and cradles; photographs.
7. Maintenance of Wood-stave Pipe in Hydro-electric Practice, by Byron E. White. Power, Vol. 60, pp. 794–6, 1924. Methods of installing and maintaining wood pipe. Failure cases are cited; photograph.
8. Improper Backfill Causes Failure of Wood-stave Pipe Lines, by Byron E. White. Power, Vol. 60, pp. 1019–20, 1924. Cases cited; photographs.
9. Design as a Factor in Upkeep of Wood-stave Pipe, by Byron E. White. Power, Vol. 61, pp. 139–142, 1925. Points regarding design of connections, cradles, provision of vents, etc., cuts and photographs.

CHAPTER XXII

CONCRETE PIPES

BY JOEL D. JUSTIN

250. General.—A general outline of the purpose and use of pipes in hydroelectric developments and a discussion of those features of pipes that are common to all types of conduits are given in Chapter XVII. The pipe between the forebay and the surge tank is commonly called the " pipe line " and that between the surge tank and the turbines is called the " penstock."

It is very seldom that concrete pipes have a place in water-power development except in localities where the hauling charge for plate-steel or wood-stave pipe is excessive, or where, for some inaccessible place or part of the work, a permanent type of construction is desired.

For successful operation, the size of the pipe, for a given discharge, may vary between wide limits; but there is usually only one size that will make for greatest economy of design. The reader is referred to Sec. 87 for the general theory of economic design, and to Sec. 183 for its application to the special case of conduits.

Usual velocities in concrete pipes are 8 to 12 ft. per second, but much higher velocities can be used without injuring the concrete.

There is practically no limit to the size of concrete pipes. They have been used in all sizes up to about 25 ft. in diameter.

Concrete pipes may be cast in place, or they may be made in pre-cast sections at some convenient place and transported to the site.

251. General Design.—Methods of determining the loading, or maximum internal pressure, for closed conduits, and the general equations for stress in pipes are given in Sec. 184. That section also gives recommended working stresses for closed conduits in percentages of the elastic limit of the steel. An exception to this recommendation should be made, however, for the stress in the reinforcing steel in concrete pipes, as the stress, if too great, will result in hair cracks in the concrete of sufficient size and number to cause leakage. It is therefore recommended that the working stress for the greatest assumed loading be not more than 12,000 to 14,000 lb. per square inch.

Reinforcing steel for reinforced concrete pipes should fulfil the requirements of Specification *A*-15-14 of the American Society for Testing Materials, for " Billet-Steel Concrete Reinforcement Bars of the Structural Steel or Intermediate Grade," the former being preferred. Deformed bars or cold-twisted bars should be used.

The concrete is usually mixed in the proportion of 1 : 2 : 4 for heads up to about 100 ft., and richer mixtures for higher heads.

Large concrete pipes built in place have not been used extensively for heads greater than 70 to 100 ft.[1]

Proper grading of materials, care in placing the concrete, and the addition of a small amount of lime to the cement are practices recommended as tending to provide impervious concrete. The concrete should be poured in cold weather to prevent shrinkage cracks due to temperature shrinkage. Expansion joints are not considered necessary, provided ample longitudinal reinforcement is used. About one-quarter of 1 per cent of reinforcement at the sides and top has been used with success.

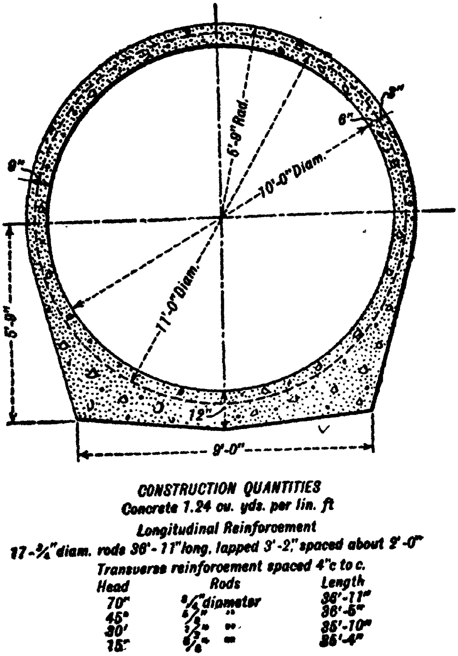


Fig. 322.—Concrete Pipe for Siphon No. 12. Los Angeles Aqueduct.

Figure 322 is a typical example of a large concrete pipe built in place.

There is, under many conditions, a material economy in using pre-cast concrete pipe. Such pipe is now used in sizes up to 9 ft. in diameter. In the use of pre-cast concrete pipe, unless special means are adopted, there is difficulty in obtaining smooth and watertight joints. In several types now on the market, this difficulty has been overcome, and pre-cast reinforced concrete pipe is now used on all heads up to 250 or 300 ft.

Such pre-cast concrete pipe is generally manufactured near the place where it is to be used. But in cases where a reinforced concrete pipe plant has been established, shipments often cover a radius of several hundred miles.

Steel forms are used, and the reinforcement generally consists of longitudinal and circumferential rods or of wire mesh. Such pipe is used for low and medium heads up to about 130 ft.

Sometimes reinforced concrete pipe is made by the centrifugal process. In this process the mold into which the concrete has been poured is revolved at a high rate of speed. This gives a very dense reinforced concrete of high strength, guaranteed by the manufacturer for heads up to 250 ft.

For higher heads, a composite pipe, consisting of a welded steel cylinder

[1] Experiments by J. H. Quinton, described in U. S. Geological Survey, "Water supply and Irrigation Paper No. 143," seem to indicate that heads greater than this are undesirable unless special precautions are taken.

surrounded inside and outside by reinforced concrete, has recently been developed by the Lock Joint Pipe Company. This pipe the manufacturers guarantee for heads up to 300 ft. The usual guarantee on leakage, for the different classes of pre-cast reinforced concrete pipe which have been described, is that the leakage will not exceed 225 gallons per inch of diameter per mile of pipe per day. This is quite a usual requirement for cast-iron water mains. Thus, for a pipe line 8 ft. in diameter and 1 mile long, the permissible leakage under the guarantee would be 0.000335 cu. ft. per second.

The pipe is generally made up in sections 12 ft. long for pipe up to about 72 in. in diameter, and in 8- to 10-ft. lengths for pipe of larger diameter.

Figure 323 shows a type of joint in which water-tightness is secured by means of a copper sealing strip which extends from the end of one section of pipe into the mortar joint. The mortar joint is bound to the next section of

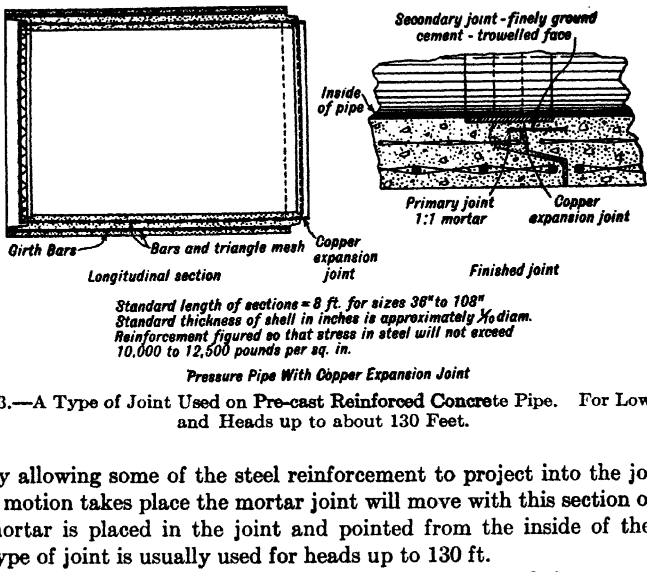


FIG. 323.—A Type of Joint Used on Pre-cast Reinforced Concrete Pipe. For Low Heads and Heads up to about 130 Feet.

pipe by allowing some of the steel reinforcement to project into the joint, so that if motion takes place the mortar joint will move with this section of pipe. The mortar is placed in the joint and pointed from the inside of the pipe. This type of joint is usually used for heads up to 130 ft.

In Fig. 324 is shown a type of joint in which heavy steel rings are molded into the ends of each section of pipe.

When the pipes are placed together and the bell and spigot ends telescope each other, a wedge-shaped cavity, with its large end towards the inside of the pipe, is formed. Into this cavity is placed an endless wedge-shaped fiber-filled lead gasket, which is calked from the interior of the pipe and which can be done at any time after the pipe has been laid. After the lead has been calked the interior groove around the circumference of the pipe is filled with mortar, making a perfectly smooth and continuous surface.[1]

This type of joint is used, for the higher heads, for the composite pipe containing a steel-welded cylinder up to heads of 300 ft.

[1] From a catalog of Lock Joint Pipe Co. of Ampere, N. J.

In a type of joint somewhat similar to that described above, the bell and spigot ends are formed by a cast-iron section. Water-tightness is secured by using a lead hemp-filled gasket similar to that described above. This is placed in the bell, and the spigot end jacked into it to secure a tight joint.

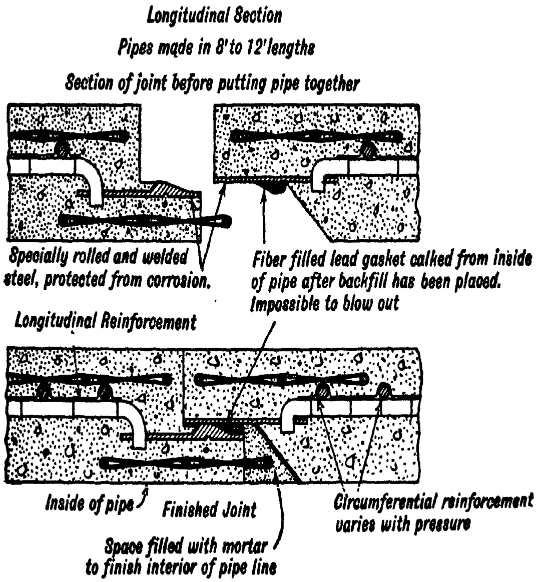


FIG. 324.—A Type of Joint as Used for Composite Reinforced Concrete Pipe for Heads up to 300 Feet.

This type of joint is used by the Lock Joint Company on their centrifugally molded pipe, for heads up to 250 ft., and is also used on the other forms of pipe under some circumstances.

CHAPTER XXIII

TUNNELS

By William P. Creager and Joel D. Justin

252. General.—A general outline of the purpose and use of tunnels in hydro-electric developments and a discussion of those features of tunnels that are common to all types of conduits are given in Chapter XVII.

253. Loading.—Methods of determining the internal loading, or maximum internal pressure, for closed conduits are given in Sec. 184. Ordinarily, the internal loading in tunnels requires consideration only where the weight of the covering is insufficient to balance the internal pressure. In such cases the tunnel lining must be reinforced.

254. Determination of Size.—For successful operation, the size of the tunnel, for a given discharge, may vary between wide limits; but there is usually one size that will make for greatest economy of design. The reader is referred to Sec. 87 for the general theory of economic design and to Sec. 183 for its application to the special case of conduits. Practically any velocity consistent with economic design may be used without damage to the tunnel lining.

255. Preliminary Investigations.—Tunneling through firm, hard rock is almost always very much cheaper than tunneling through soft or disintegrated rock or earth. This is because of the expense and delays necessitated by the bracing and timbering required for tunnels in earth and soft or disintegrated rock. Thus the material to be desired for a tunnel location is just the opposite to that which would be desired for any excavation at the surface, such as a canal excavation, for instance. The encountering of an area of disintegrated rock or earth on the line of a tunnel may result in increasing the cost per foot for this section of the tunnel to a figure several times as great as the cost of driving the rest of the tunnel through firm rock. For this reason it is especially important that the subsurface investigations along the proposed alinement of a tunnel be made in a very thorough manner. The subsurface explorations should be made by means of diamond drills or shot drills.

No general rule can be given for the spacing of the holes along the proposed alinement; but they should be so close together that there will be no reasonable doubt as to the nature of the material which will be encountered between the holes. Topography and surface indications will generally indicate how far apart the holes should be placed. For instance, if a tunnel is to pass under a hill that consists of exposed solid granite ledge, it would usually be foolish to drill any holes at this point as one may usually safely

assume that the same character of material continues to an indefinite depth. On the other hand, if a hill is covered with soil a number of holes should be put down. It is especially important to put down holes in gullies and depressions, for they are sometimes the sites of faults and fractures where areas of disintegrated rock may be located.

Holes should frequently be drilled to a considerable depth below the proposed invert of the tunnel, as, after all the information is obtained, it may be found that firmer rock could be obtained by driving the tunnel at a greater depth. If the information obtained for a particular alinement proves to be not very favorable, additional possible alinements should be investigated and the most favorable selected.

The proper placing of the holes and the interpretation of the investigations require considerable experience in such work and a working knowledge of geology.

256. Tunneling in Earth.—Tunnels may be driven through almost any material occurring in nature; but the methods used and the costs differ radically.

Shield Method.—Subaqueous tunnels through soft ground are generally driven under compressed air, by means of a steel shield which is forced ahead by hydraulic jacks resting on the flanges of the cast-iron or steel segments that form the lining of the tunnel. Tunneling under these conditions is very expensive, and, so far as the authors know, it has never been necessary to use this method on a tunnel driven for water power purposes.

Poling-board Method.—This method of tunneling is applicable for use in ordinary soil or where a tunnel runs out of ledge rock and into loose glacial material. Often a water-power tunnel must be started by this method and so continued until ledge rock is reached in the hillside.

As indicated in Fig. 325, pointed boards from 2 to 4 in. thick and 6 to 10 ft. long are driven forward and slightly upward over the top of each cap as each successive bent of timbering is placed. The poling boards are kept driven firmly into the face of the heading as it is excavated, and as soon as enough has been taken out to permit of placing another sill another bent is erected and the process continued. If there is a tendency for the sides to slide in, poling boards are used at the sides in the same manner, the struts being braced to cap and sill to form a rigid frame.

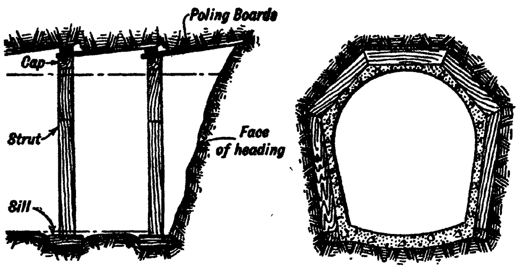


FIG. 325.—Poling Board Method of Soft Ground Tunneling.

257. Tunneling in Rock.—Tunneling in firm ledge rock, which does not require timbering, is generally much cheaper than tunneling in any other material. Methods differ materially with the nature of the rock and with local conditions.

Simple Heading Method.—In driving a tunnel not over 10 ft. high, the work is generally conducted as a simple heading operation. Fig. 326 is a cross-section of the Lower Fielding Tunnel, Iron Mountain, California, showing the general arrangement and direction of the drill holes as used in the advance of the tunnel. The rock was a hard felsite (a fine-grained igneous rock). About one-third of the total length of the tunnel was timbered. The tunnel was 8 ft. 6 in. high and 9 ft. 6 in. wide. Less than 3 lb. of dynamite was used per cubic yard of excavation.[1]

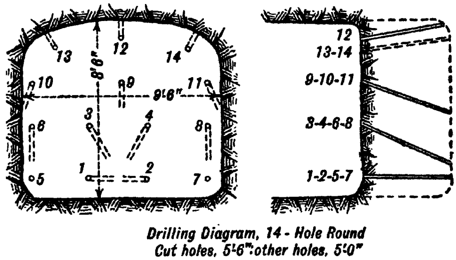


Fig. 326.—A Typical Simple Tunnel Heading.

The general operations of drilling, shooting, and mucking a heading are quite similar in all cases. The drills used are almost always of the compressed-air-operated percussion type. The heavier drills are mounted on columns or tripods for use. For light work and for trimming and breaking up muck, a lighter drill, known as the Jack hammer, which can be handled by one man without the use of column or tripod, is very useful and popular. Electric drills have been used, but they have not proved entirely successful as a substitute for the air-operated percussion type.

The drills are set in the heading on drill columns, which are held firmly in place by jacking against floor and roof. In some cases the columns are horizontal and are held in place by jacking against the ribs. The holes are then drilled about as indicated in Fig. 326. Then, while the columns and drills are being removed to a place of safety, the holes are blown out with compressed air, loaded, and tamped. The shooting should be done by a firing battery or else by means of dry cells. Firing fuse should not be used except for isolated holes. It is frequently very important to shoot a number of holes simultaneously, and one can never be entirely sure of accomplishing this when using fuse. The use of a firing battery is also much safer.

The cut holes, as Nos. 1, 2, 3, and 4 in Fig. 326 are always fired first. This blows out a wedge, enabling the other holes, when fired, to break down the rock. The sequence of firing the other holes varies with the nature of the rock and the judgment of the superintendent. Holes 5, 6, 7, and 8 would probably be fired next, and then the remaining holes probably in a single shot.

The depth to which the holes are drilled varies, in general, from 5 to 11 ft.

As soon as men can get into the heading, the muckers are put to work loading the rock that has been shot down, on to cars; after this it is hauled out of the tunnel. Effective mucking machines for handling this material and loading it on to cars are now in use, and give excellent service in many cases. They are made in sizes that can be used in quite small tunnels. In very large tunnels, steam shovels operated by compressed air or electricity are frequently used for mucking. As soon as the mucking is sufficiently far advanced, the

[1] W. C. Hammatt, p. 750, Trans. Am. Soc. C. E., Vol. LXXV (1912).

drills are set up again and the sequence of operations repeated. Usually from one to three complete operations are obtained per twenty-four hours, but soft ground and timbering may hold up the work at any time for several days. A net average progress of from 300 to 400 ft. per month is usually considered good work, and from 200 to 300 ft. per month average work.

Top-heading Method.—In driving tunnels of large size, say 20 ft. in diameter or more, there is considerable variation in the sequence of operations. In America, the use of the top heading is the prevailing method. A heading or smaller tunnel, similar to that shown in Fig. 327, is first driven ahead of the main tunnel at the top of the section, in the manner indicated in Fig. 326. The height of this heading is usually from 8 to 10 ft. In very wide tunnels two top headings are sometimes driven side by side leaving a rib or dividing wall of rock between, which is afterwards removed. The top heading is kept about one shift ahead of the bench, so that very little muck has to be handled over the bench. The main mucking operations are carried on from the finished grade back of the bench by mucking machines or steam shovels. The bench is drilled by tripod drills, and this part of the excavation is usually much cheaper than the heading excavation. When timbering is required it should immediately follow the shooting of the bench. Sometimes temporary timbering is required in the heading to protect the men working there.

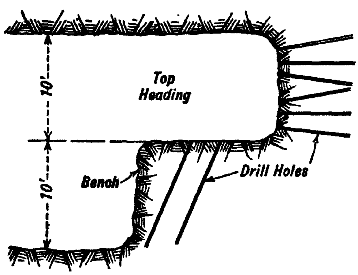


FIG. 327.—Longitudinal Section Showing Top Heading and Bench.

258. Shape of Section.—The shape of the section through absolutely stable material may be whatever proves the most economical from a construction standpoint. Absolutely stable material, however, is seldom encountered in nature, and the shape of the section must be such that the lining affords the best resistance to the external pressures. This varies with the nature of the materials through which the tunnel is driven, and no standards have been generally adopted.

The roof of the tunnel is almost invariably supported by a semi-circular arch. For hard rock having no tendency to lateral movement, vertical sides may be adopted as in Fig. 330, and the bottom may be horizontal.

Firm earth or soft rock tunnels having a slight lateral pressure are provided with a horseshoe-shaped section as in Fig. 328.

Tunnels through very soft earth, and earth tunnels subjected to unbalanced external water pressure, must be circular or nearly so. It must be remembered that tunnels are frequently emptied and the elevation of ground water is often much increased by leakage or seepage from the reservoir.

In some instances drains have been provided in the lining to relieve the external pressure when the tunnel is emptied. Such drains are, however, objectionable if they may cause excessive leakage when the tunnel is full.

Tunnels in rock subjected to unbalanced eternal water pressure need not

be circular, but the lining must be figured to withstand such pressure by arching. A horseshoe section with an inverted-arch bottom is in common use for moderate external pressure.

The bottom of an earth tunnel must be designed to support the weight of

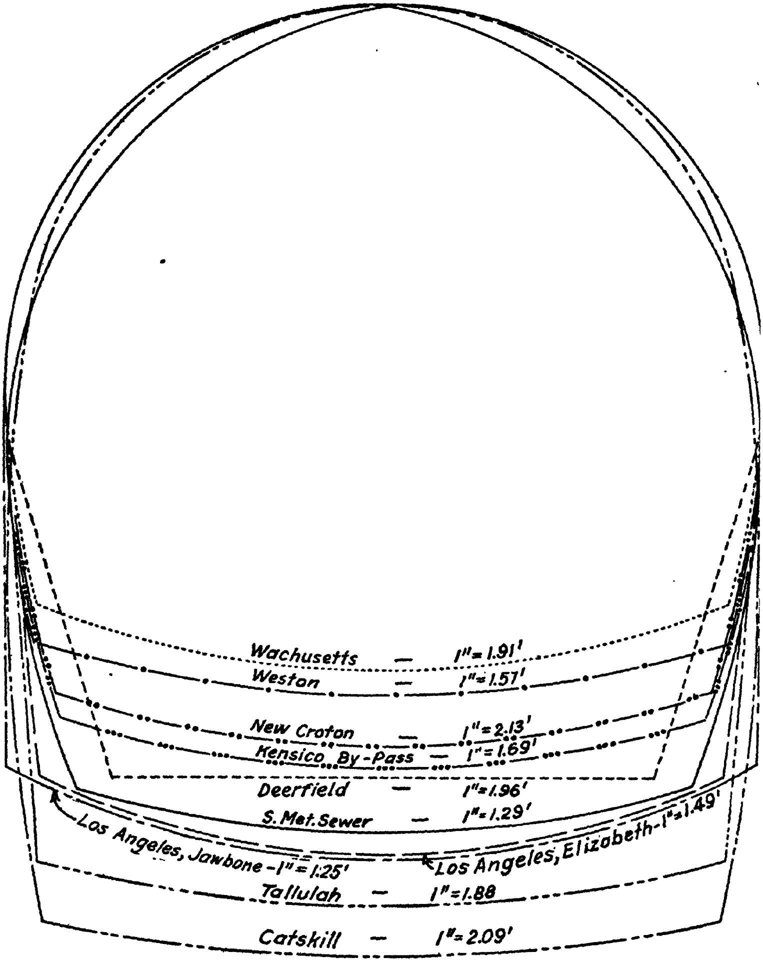


FIG. 328.—Composite Tunnel Sections Plotted with Equal Width.

the tunnel and the top load, without settlement. It must, therefore, be very thick or shaped as an inverted arch to distribute the side-wall loads over the whole base.

Tunnels reinforced for internal pressure must be of a shape that will permit

of circular reinforcement; but the outside and inside of the lining may vary from a circle for the sake of economical construction, provided the circular reinforcement is well embedded.

In order to provide reasonable room for workmen and machinery, a height of excavation of 6 or 7 ft. must be adopted, because a smaller tunnel would cost more per linear foot on account of increased difficulty in its construction.

259. Tunnel Lining.—It is the usual practice to line tunnels with concrete, although in the past many have been lined with brick. The purpose of the lining is threefold:

1. To obtain a smooth surface which will give a minimum loss of head.
2. To increase the structural safety of the tunnel.
3. To prevent loss of water by seepage through seams in the rock.

If the depth of cover is insufficient to counterbalance the hydrostatic pressure within the tunnel with a fair margin of safety, the lining should be reinforced to take care of the excess hydrostatic pressure. It is not usually necessary to do this because it is generally practicable to keep the tunnel at a depth where it will be safe without reinforcing the lining.[2]

In a number of cases it has been considered economical to line the tunnel even in firm hard rock, where there was no danger of falls. That is, it was considered that a lined tunnel would be cheaper than an unlined tunnel of equal capacity. While the evidence is not sufficiently conclusive to permit of laying down a general rule, it will be found that most important pressure tunnels are lined. Each particular case should be thoroughly investigated before deciding whether or not to line a tunnel. Where a tunnel is timbered to support the roof, concrete lining is necessary if permanent structural safety is desired. In certain other cases the rock is so seamy that lining must be resorted to in order to prevent serious loss of water.

The thickness of lining required depends upon the nature of the material to be supported and the method of construction. As indicated in Fig. 329, the " neat line " is the prescribed line within which no rock or timbering must project. Since it is impossible to excavate exactly to prescribed lines, the " line of average excavation " is the average line to which the material is actually excavated; this line also fixes the " average thickness " of concrete lining. The " net thickness " of lining should have a minimum of 4 to 8 in., depending upon the size of the tunnel. The " overbreak," or excess material removed beyond the prescribed neat line, for rock tunnels, has a minimum of about 8 in. and may be several feet if the tunnel is heavily timbered. Examples of overbreak are given in Table LIII. Overbreak averages about 12 in. for stable rock tunnels. Thus the average thickness of lining of rock tunnels is required by practical considerations to be at least about 12 in. and averages 16 to 20 in. Therefore, it is seen that, unless the tunnel is very large or the rock quite unstable, the minimum thickness of lining which it is practicable to build is of ample strength.

Only under exceptional conditions can earth tunnels be excavated without timber supports. Under such conditions, however, the amount of overbreak would be very small. Timber supports are frequently not removed and,

[2] See also Sec. 258.

as they cannot be counted on to contribute to the strength of the lining unless continually saturated, their area must be deducted from the average thickness of lining.

It is impossible to estimate exactly the external loads to which the tunnel lining will be subjected. Therefore, the engineer must be guided by what experience has proved to be adequate under similar conditions.

For stable rock, a net thickness of 4 in. for small sizes to 8 in. for large tunnels has been used.

For unstable rock requiring no timbering or timbering which is removed

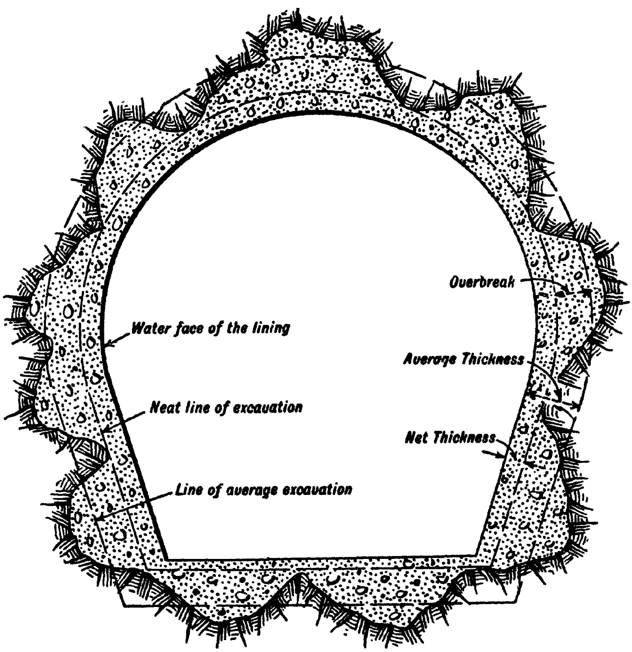


Fig. 329.

before the lining is placed, 8 to 12 in. is common. This, together with 8 to 10 in. of overbreak, results in an average thickness of 16 to 24 in.

For unstable rock tunnels in which the timbering is not removed, as in Fig. 330, a thickness of 4 in. is sufficient. This will result in a total thickness of 4 in. plus, say, 11 in. for timbering, plus, say 10 in. for overbreak beyond the lagging, or about 25 in. average thickness less the area of the bents.

The same condition for earth tunnels requires 8 to 12 in. net thickness, as the overbreak beyond the lagging is negligible. The average thickness would then be 19 to 23 in.

It is fairly certain that the thickness of lining frequently adopted for tunnels

is excessive. This is well illustrated by the fact that a "fall" of rock in a heading will almost always block itself within a height equal to the width of the heading. For example, assume that a heading 12 ft. wide is being driven with the effort made to keep the roof horizontal. If an area of very poor rock

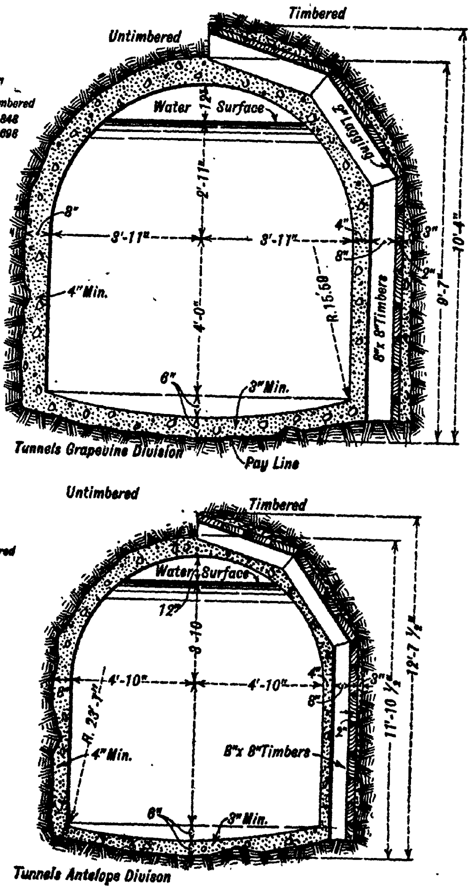
Construction Quantities

Per lineal foot of tunnel normal section

	Unit	Timbered	Untimbered
Excavation to pay line	Cu. Yd.	3.514	2.848
Concrete in lining	" "	.945	.698
Timbers	B. M.	32.0	
Spreaders	" "	5.0	
Shoulder Braces	" "	7.0	
Lagging	" "	47.0	

Hydraulic Properties.

$S = .0017$
$Q = 484.18$
$V = 7.87$
$A = 54.462$
$P = 22.499$
$R = 2.42$
$n = .014$
$c = 124.30$



Construction Quantities

Per lineal foot of tunnel normal section

	Unit	Timbered
Excavation to pay line	Cu. Yd.	6.12
Concrete in lining		1.73
Timbers	B. M.	4.25
Spreaders		6.0
Shoulder Braces		7.0
Lagging		80.0

Hydraulic Properties.

$S = .0005$
$Q = 434$
$V = 4.94$
$A = 87.96$
$P = 29.31$
$R = 3.00$
$n = .014$
$c = 127.4$

FIG. 330.—Standard Types of Tunnel in the Grapevine and Antelope Valley Divisions. Los Angeles Aqueduct.

is encountered, the resulting fall of rock will seldom extend more than 12 ft. above the theoretical roof line.

There is, however, good reason for this, as the nature of the loading is uncertain and a faulty lining would prove very expensive to a going plant.

Unless external water pressure is expected, the invert of a rock tunnel may have a thickness which is only sufficient to smooth up the surface. Earth

tunnel inverts, however, must be designed to distribute the arch load and prevent settlement.

The lining must, of course, be designed to withstand all external water pressure, or else drained as explained in Sec. 258.

Unless the overbreak is very excessive, causing the formation of large pockets outside of the required thickness of lining, the concrete lining is placed in all cases in direct contact with the rock surface. In exceptional cases where the overbreak is very excessive, the tunnel lining is formed to the required minimum thickness and the cavities left on the outside are then "dry-packed" with loose rock or "spalls." Then after the concrete lining is completed, these areas are grouted through holes in the concrete lining.

It is quite difficult to fill completely with concrete the space over the top of the arch. Frequently this space is filled with loose stone and grouted. If the space is to be continually full of water, as in the case of an earth tunnel from the forebay to and through a rock barrier, or if ground water is high enough, cord wood may be used for such filling. Care should be taken, however, that if the space is not grouted, it is broken by frequent water-tight cut-offs to prevent free passage from the forebay or from tunnel leakage.

Ordinarily, a concrete proportioned for an ultimate compressive strength of 1500 lb. will answer the purposes of tunnel lining. When a high degree of water-tightness is required, or when the external pressures on the tunnel are very severe (owing to areas of loose or disintegrated rock), richer mixtures giving higher ultimate strengths are sometimes used. The mixtures used vary all the way from 1–2–4, to 1–3–6–; and, as the lining is usually thin, a surplus of mortar is frequently used to insure plasticity and the filling of all irregularities. When the minimum thickness of lining is 6 in., the maximum size of aggregate should be limited to 1 in. When the minimum thickness of lining is 12 in., the maximum size of aggregate should be limited to 1½ in.

260. Depth of Tunnel.—In general, the tunnel alinement should be chosen so as to get below faulted or broken or disintegrated areas, or else to miss them by being located to one side. Most ledge surfaces are likely to be quite badly broken up by weathering cracks near their tops. Consequently, it is sometimes necessary to go at least 50 or 60 ft. below the rock surface in order to secure sound rock.

With tunnels operating under pressure, it is necessary to go to such a depth that the weight of the rock above the top of the tunnel is sufficient to counterbalance the hydrostatic pressure. This is very important, as there are a number of cases on record where pressure tunnels have failed because of unbalanced hydrostatic pressure.

At Sand Canyon of the Los Angeles Aqueduct there are two inclined pressure tunnels leading to a steel pipe, the whole forming an inverted siphon across the canyon. These tunnels were driven through hard gray granite having occasional seams. The maximum head was to be 455 ft. These tunnels are lined with 12 in. of concrete and are 9 ft. in diameter. The lengths were 631 ft. and 638 ft. Before going into actual operation they were subjected to a pressure of 60 to 70 lb. and failed, although there was apparently 83 ft. of covering over the tunnel. Mr. Mulhulland, Chief Engineer of the

project, in commenting on the failure says: " The rock and tunnel were ruptured on these two different occasions (two tests were made) and it is believed that in the first rupture the overlying prism itself was lifted and set down again in a position that was different by 2 in.

These two pressure tunnels were later abandoned, at least in part, and steel pipe substituted.

Reliance should be placed only on the weight of the overburden above the roof of the tunnel; and no dependence whatever should be placed on the shear value of the ledge rock, even if it is unseamed granite ledge. No dependence should be placed on the concrete lining unless it is reinforced.

Tunnels through earth should not be used for pressure tunnels unless reinforced, except where it is certain that the elevation of ground water at all times corresponds to the internal pressure.

In cases where it is uneconomical or impossible to balance the hydrostatic pressure of the tunnel by weight of overburden, and also near portals, the tunnel lining is reinforced to take the full internal pressure. This is done by using rings of reinforcing steel, hoops made up of angle iron, etc. In some cases, where the pressure is high, riveted steel plate lining is used, backed up by concrete on the outside of the steel shell to fill the space between the rock and the steel plate, and lined with concrete on the inside to give a smooth surface and preserve the steel.

261. Overbreak and " Pay-line."—*Rock Tunnels.*—In preparing a preliminary estimate of the excavation that will be required for a tunnel in rock, it is necessary to include a proper allowance for overbreak. As here used, overbreak means the rock that is actually excavated in addition to that which is included within the required neat line of excavation of the tunnel. Thus, as in Fig. 329 the cross-sectional area of a concrete-lined tunnel, that is, the water-carrying area, might be 154 sq. ft., while the area to the line beyond which no rock is allowed to project, called the neat line of excavation, might be 200 sq. ft., and the area enclosed by the line to which the excavation actually breaks, say, 250 sq. ft. The percentage of overbreak is, then, 50/200, or 25 per cent. The percentage of overbreak is sometimes, but incorrectly, stated as the percentage of overbreak over the tunnel section, or water-carrying area.

The actual amount of overbreak depends very largely on the nature of the rock and the skill with which the drilling and shooting are done. For rock tunnels requiring timbering that cannot be removed before the lining is placed, the overbreak is greater by the space outside the neat line to be provided for the timbering.

In unit-price contract work, it is necessary to limit the overbreak because, with excessive overbreak, the rock can be removed and the thicker lining placed at less cost per cubic yard. Therefore, it is customary, in writing unit-price contracts, to provide outside limits to the excavation and concrete, called the " pay-lines," beyond which no excavation or concrete will be paid for.

For untimbered tunnels, the pay-lines for excavation and concrete are usually identical. They are usually placed a constant distance outside the neat line of excavation and are fixed as near as possible to the probable line

of average actual excavation, Fig. 329, which would be consistent with reasonable care.

Table LIII shows the average overbreak "*d*," for a number of tunnels. It will be noted that, by eliminating a few exceptional cases, this distance for untimbered rock tunnels averages about 12 in. and this figure was used by the New York City Board of Water Supply as a standard for its specifications and estimates.

For rock tunnels having timbering left in place, the pay-line for excavation at the sides and top is moved out a distance equivalent to the space estimated to be required for the timbering. This space averages about 12 in., making the average total distance from the neat line about 24 in. at the sides and top. However, this space varies greatly according to the size of the tunnel and the nature of the rock.

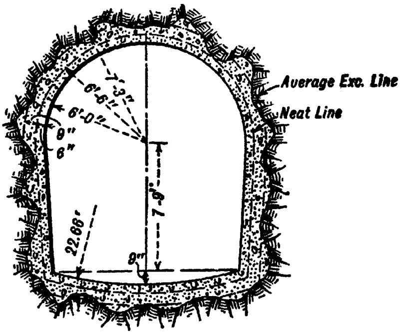


FIG. 331.—Tallulah Falls Tunnel. Georgia Railway & Power Co.

The pay-line for concrete in timbered rock tunnels is inside the pay-line

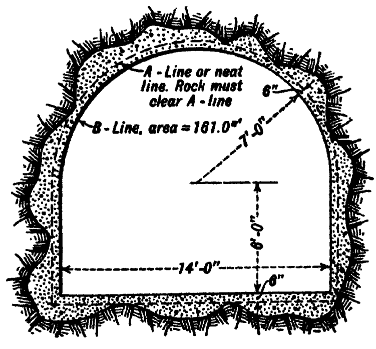


FIG. 332.—Davis Bridge Tunnel. New England Power Co.

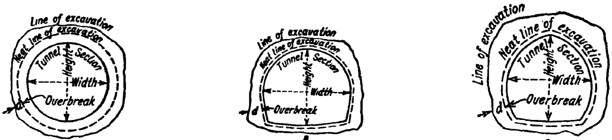
Water cross-section area of tunnel as designed	161.	sq. ft.
Average water cross-section area	162.5	sq. ft.
Actual minimum water cross-section area	160.5	sq. ft.
Actual maximum water cross-section area	163.7	sq. ft.
Actual cubic yards of solid excavation per linear foot	7.85	sq. ft.
Minimum thickness of concrete lining permitted	6	inches
Average thickness of concrete lining per linear foot	12	inches
Average cubic yards of concrete lining per linear foot	1.83	

for excavation by a distance corresponding to the estimated volume of the timbering required.

Overbreak is far from constant and should be based on the nature of the rock. It is considered that modern methods and a greater knowledge of blasting will result in a smaller overbreak than the average of 12 in. given

TABLE LIII

Studies in Overbreak in Excavation for Tunnels



A B C

Number	Shape of Section	Dimensions: Tunnel: Height	Dimensions: Tunnel: Width	Dimensions: Neat Excavation: Height	Dimensions: Neat Excavation: Width	Areas, Square Feet: Tunnel Section	Areas, Square Feet: Neat Section	Areas, Square Feet: Amount Excavated	Per Cent of Total Excavated Over Tunnel Section	Per Cent of Overbreak Over Neat Section	Overbreak "d"	Nature of Material Excavated	
* 1	B	12′ 2″	13′ 2″	12′ 11″	13′ 10″	130	150	192	47.8	28.0	0.90	Rock—Diorite—Much jointed and slicken sided	U
† 2	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	109	133	164	50.3	23.2	0.75	Rock—Hard massive granite, numerous tissues	U
* 3	B	9′ 3″	10′ 0″	10′ 0″	10′ 8″	76	87	126	65.7	44.7	1.07	Rock—Granite generally coarse grained	U
* 4	B	9′ 3″	10′ 0″	12′ 0″	12′ 4″	74	117	157	112.2	34.1	0.99	Earth—Stratified clay and boulder clay	E
* 5	B	9′ 3″	10′ 0″	10′ 0″	10′ 8″	76	88	127	67.2	44.4	1.07	Rock—Granite coarse grained	U
* 6	B	9′ 3″	10′ 0″	10′ 0″	10′ 8″	74	85	122	65.0	44.2	1.07	Rock—Granite coarse grained	T
† 7	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	106	131	161	51 8	22 8	0.74	Rock—Black graphite slate	U
† 8	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	103	130	165	60.2	26.9	0.85	Rock—Diorite generally massive and laminated	U
† 9	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	105	130	173	64.8	33.1	1.03	Rock—Schist ranging from rotten to solid and massive	U
‡10	A	8′ 6″	8′ 6″	10′ 6″	10′ 6″	54	82	119	120.4	45.1	1.06	Rock—Gniess micaceous	U
†11	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	107	133	179	67.5	34.6	1.08	Rock—Schist—Hard gritty from massive to laminated	U
†12	B	10′ 10″	12′ 2″	12′ 2″	13′ 6″	108	131	172	59.2	31.6	0.98	Rock—Schist—Shaly and stratified	U
*13	B	12′ 2″	13′ 2″	12′ 11″	13′ 10″	131	145	187	42.7	29.0	0.93	Rock—A complex quartzite and diorite	U
*14	B	12′ 2″	13′ 2″	12′ 11″	13′ 10″	127	142	180	41.8	26.8	0.86	Rock—Granite close grained and massive	U
§15	C	9′ 2″	8′ 3″	10′ 6″	9′ 7″	59	79	116	96.6	47.5	1.02	Conglomerate Roxbury pudding stone	U
§16	C	7′ 0″	6′ 6″	7′ 8″	7′ 2″	35	43	78	122.8	81.6	1.24	Conglomerate Roxbury pudding stone	U
§17	C	9′ 9″	9′ 3″	11′ 5″	11′ 3″	70	101	144	105.8	42.6	1.08	Rock—Felsite very hard and flinty	U
§18	C	7′ 0″	6′ 6″	7′ 8″	7′ 2″	35	43	74	111.2	72.5	1.12	Conglomerate Roxbury pudding stone	U
*19	B	12′ 2″	13′ 2″	12′ 11″	13′ 10″	127	142	191	50.4	34.5	1.11	Rock—Diorite gray to black solid	U
§20	A	9′ 0″	9′ 0″	10′ 4″	10′ 4″	61	81	109	78.8	34.8	0.83	Conglomerate Roxbury pudding stone	U
§21	C	7′ 0″	6′ 6″	8′ 6″	8′ 6″	35	55	81	131.0	48.3	0.94	Boulder clay or hardpan half timbered	E
§22	C	7′ 0″	6′ 6″	7′ 8″	7′ 2″	35	42	94	168.3	123.6	1.76	Trap Rock—Excavation full sized, no bench	U
*23	B	9′ 3″	10′ 0″	10′ 0″	10′ 8″	80	89	186	132.5	110.0	2.40	Rock—Coarse grained granite timbered	T
‡24	A	14′ 0″	14′ 0″	16′ 0″	16′ 0″	153	204	254	65.9	24.4	1.45	Rock—Hard gniess	U
‡25	A	10′ 6″	10′ 6″	13′ 2″	13′ 2″	88	148	222	152.1	50.4	1.48	Rock —Limestone hard and white	U
‡26	A	10′ 6″	10′ 6″	13′ 2″	13′ 2″	89	144	222	149.5	54.2	1.59	Rock—Hard gneiss	U
‡27	B	13′ 7¼″	13′ 6″	15′ 6″	15′ 7¼″	156	207	282	80.8	36.1	1.31	Rock—Hard gneiss	U
‡28	B	13′ 7¼″	13′ 6″	15′ 6″	15′ 7¼″	155	204	265	70.9	29.8	1.09	Rock—Limestone hard and compact	U
‡29	B	13′ 7¼″	13′ 6″	15′ 6″	15′ 7¼″	154	203	274	78 0	35.0	1.25	Rock—Gniess	U
¶30	B	13′ 0″	14′ 0″	14′ 0″	15′ 0″	161	185	212	31.7	14.0	0.50	Rock—Mica schist with occasional quartz veins	U
Average											1.118		
Average omitting No. 23											1.075		
Average omitting Nos. 22, 23, 25 and 26											1.013		

* Weston Aqueduct.
† Wachusett Aqueduct.
‡ New Croton Aqueduct.
§ South Metropolitan High Level Sewer.
¶ Davis Bridge.

U = Untimbered Rock.
T = Timbered Rock.
E = Timbered Earth.

hereinbefore for untimbered tunnels. It will be noted that the overbreak for the Davis Bridge tunnel was only 6 in.; but this is exceptional, even with the best methods and easiest excavation.

A firm igneous or metamorphic rock with few seams can usually be made to break closer to the neat lines than a sedimentary rock; and a sedimentary rock in which the strata are vertical will, other things being equal, usually break closer to the neat line than a sedimentary rock that has horizontal stratification.

Earth Tunnels.—The term " overbreak " is not used in connection with earth tunnels as in such work it is possible to excavate very closely to prescribed lines. The pay-line for excavation is at a distance from the neat line equal to that estimated to be required for the bents and poling boards. This distance varies from 12 to 18 in. depending upon the size of the tunnel.

The pay-line for concrete is inside the pay-line for excavation by a distance corresponding to the estimated volume of timbering required.

262. Adits and Shafts.—If the proposed tunnel is long and the progress schedule severe, the cost of hauling the muck long distances underground usually requires that the tunnel be broken up into a number of different sections and worked simultaneously from a number of different headings. To reach these headings a number of construction shafts are sometimes required. Sometimes the tunnel can be reached by means of adits or drifts in the hillside, which are merely side tunnels driven from the face of the hill to the alinement of the tunnel to permit of starting additional headings. In locating the alinement of a tunnel, the ease of access for construction purposes should be given due consideration. Other things being equal, it is cheaper so to locate the tunnel that it can be reached at several points in the line by means of drifts at grade, instead of placing it away back in the hill where construction shafts of great depth might be required.

Shafts are usually either circular or rectangular in section. In cases where a considerable depth of soft ground must be penetrated before ledge rock is reached, the shaft is frequently sunk as a steel or reinforced concrete open caisson. In extreme cases, pneumatic caissons are used. Sections of the steel or reinforced concrete shaft lining are built up above ground and sunk by excavating below the bottom or cutting edge of the caisson. When the soft ground conditions are not so severe, the shaft is sunk by the poling-board method,[3] and where the material is fairly firm the earth is simply excavated and the shaft lined with lagging and braced as the work progresses.

When ledge rock is reached the method of procedure is quite similar to that used in driving a heading. After a set of drill holes have been completed, a " V " is first shot out of the center of the shaft. This provides a space for the other holes to break to when they are shot. Each time, before shooting, everything must usually be lifted out of the shaft. This usually makes shaft sinking a slower process than driving heading. For this reason, shaft sinking ordinarily costs from 1.5 to 2 times as much per cubic yard of rock taken out as tunnel excavation in similar material and of the same cross-section.

[3] See Sect. 256.

263. Drainage.—In the construction of almost all tunnels there is some seepage water which must be taken care of. In some cases the quantity of this seepage water is excessive and if it must be pumped up deep shafts, the cost of pumping may be a considerable part of the total cost of the tunnel. In such cases the use of drifts at grade has a very great advantage as the water

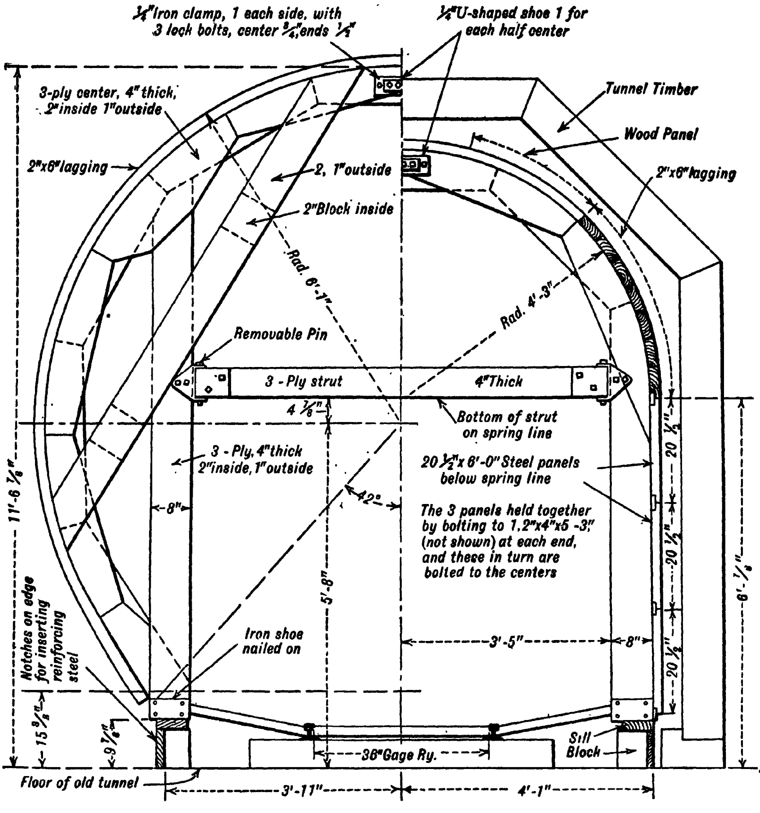


FIG. 333.—Forms for Two Types of Tunnel Lining. Kern Canyon Project, California. From Engineering News-Record, Vol. 89, p. 54, 1922.

may be allowed to run out by gravity and the cost of pumping saved. A slope of 6 in. in 100 ft. is frequently adopted.

264. Method of Placing Lining.—The invert of the tunnel lining is usually placed first, even when the tunnel is circular in section. Curved templates are set to grade and line, and the concrete, usually transported on cars, is placed between them and screeded into place to a finished surface. Placing the invert first gives a floor on which the forms can be erected and moved.

The forms are generally so designed and built that they can readily be collapsed and taken down and moved ahead. Sometimes they are so built that one section can be collapsed and moved under the section just ahead, and erected ahead of the section of forms which is being poured. This method permits of practically continuous concreting. If the tunnel is of material length, it is economical to use steel forms, but even if wooden forms are used the lagging should be covered with sheet iron in order to secure as smooth a surface as possible. No. 20 or No. 24 gage sheet iron is suitable for this

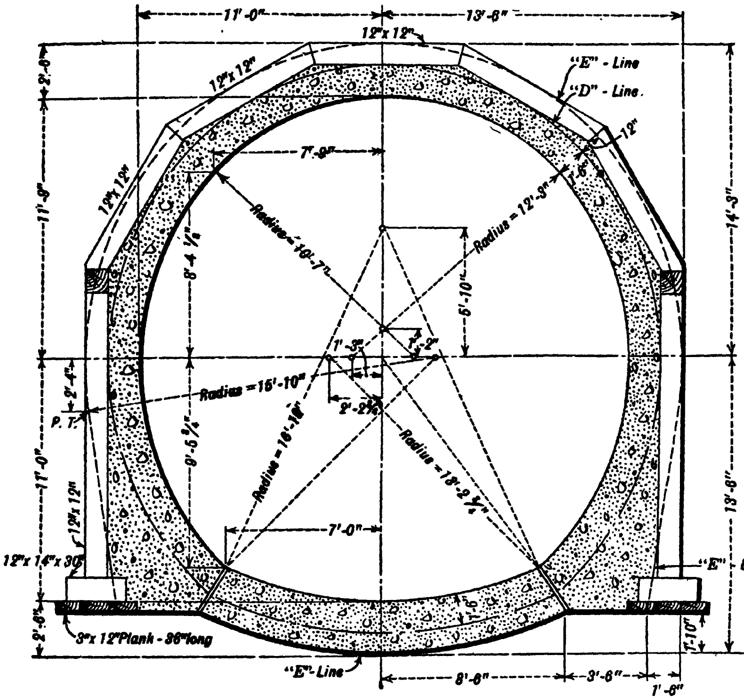


FIG. 334.—Standard Earth Section, Davis Bridge Development, New England Power Co

purpose. No pains should be spared to get the concrete surface as true to line and grade as possible, as the friction loss may thus be greatly reduced.

The pneumatic method of mixing and placing concrete is particularly well adapted for use in tunnel lining, and has been used with marked success. In the older methods of placing concrete lining, it was necessary to use relatively dry concrete in the arch, as it had to be packed back over the arch and tamped into place by hand from the side. This made the placing of the upper part of the arch a slow and laborious process. A void was almost always left between the crown of the arch and the rock, which could only be filled by grouting. By the pneumatic method the form is bulkheaded at

the ends and the concrete poured into place just about as readily in the arch as in the side walls.

By the pneumatic method the concrete is usually mixed at the surface. Sometimes the mixing is done by air entirely. In this case the concrete materials are dumped into the top of a cast drum, and air is admitted through a large number of small pipes at the bottom. This agitates the materials and thoroughly mixes them together. The door at the top of the drum is then closed and air pressure turned into the upper part of the chamber. There is a line of supply pipe from 6 in. to 10 in. in diameter, leading from the bottom of the air mixer down the shaft and to the form to be concreted. The gate connecting this pipe line to the bottom of the mixer is then opened and the charge of concrete forced through the pipe line to the point of deposit. This method requires the use of a very considerable quantity of compressed air and a large compressor installation.

To meet this objection, the Ransome people have placed on the market a modified pneumatic placing method. Under this method the concrete is mixed by mechanical means in a drum-type revolving mixer and the compressed air is used merely to force the charge of mixed concrete through the pipe system to the point of deposit. A great economy in the use of air is claimed for this system.

The principal objection to any pneumatic system of placing concrete is the great wear which takes place on the pipes used for transporting the concrete. This wear is minimized by using special steel pipe and by using long-radius bends made up of short segments made of special steel. As most of the wear is concentrated at the bottom of the pipe, the life of the pipe is increased by rotating it slightly after each few runs. The wear on the pipe line will be much greater with some aggregates than with others.

265. Ventilation.—The ventilation of tunnels during construction is a matter which does not always receive the attention that its importance requires. Tunnels are sometimes driven without any attempt at ventilation, dependence being placed on the air released at the drills together with the occasional release of air by opening a valve in the compressed air main at the heading when the air becomes very bad. A great deal of inefficiency results from the lack of proper ventilation, and it almost always pays handsomely to provide plenty of fresh air from outside at the heading. A fan located on top, near the shaft, generally supplies large quantities of air at low pressure to a supply main made of spiral riveted pipe of very thin metal. This ventilating pipe, which is usually 16 in. or more in diameter, is conducted along the tunnel to the heading. It is frequently attached to the roof in order to be out of the way. The last few sections at the heading are often composed of heavy canvas. When shooting is done the fans are stopped and the canvas section turned back out of harm's way. As soon as the shot is fired, the air is turned on and the heading quickly cleared of smoke and gases. Where there is improper ventilation, it is often a long time after a shot before men can get into the heading. With proper ventilation this delay is greatly reduced, and this saving alone will frequently pay for the cost of installing a proper ventilating system.

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CHAPTER XXIV

WATER HAMMER

By Eugene E. Halmos and William P. Creager

267. Definition.—Water hammer is defined as the change in pressure, above or below normal pressure, caused by sudden changes in the rate of flow of water. Water hammer occurs in penstocks between the forebay (or surge tank) and the turbines, and also to a less extent in pipe lines between the forebay and the surge tank, because of sudden changes in the demand for water during load fluctuations, and the pressure in these pipes is subject to large fluctuations at all points.

For a sudden decrease in load demand, the turbine gates close, and positive water-hammer pressure is created, as indicated by gradient AB, Fig. 335A. After the gate movement ceases, the positive water-hammer pressure, AB, swings to negative pressure, AC, an equal distance below static. The pressure then fluctuates between positive and negative until dampened out by friction.

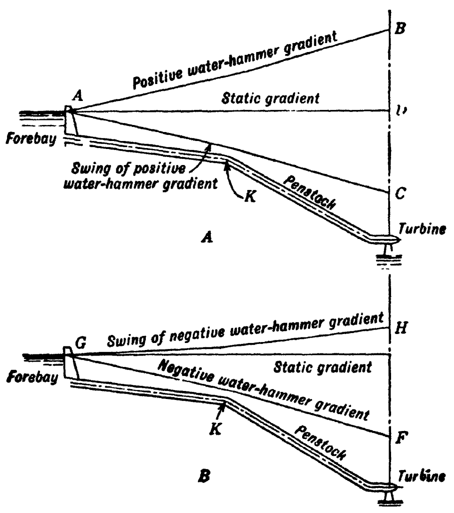

Fig. 335.

For a sudden increase in load demand, the turbine gates open, and a negative water-hammer pressure is created, as indicated by gradient GF, Fig. 335B. After the gate movement ceases, the negative pressure, GF, swings to positive pressure, GH; but the rise above static is not necessarily equal to the drop below static.

The penstock should be designed to withstand, at every point, an internal pressure corresponding to the maximum positive water-hammer pressure, AB. The negative water-hammer gradient, AC or GF, whether caused directly by gate opening or by the swing of pressure from positive to negative after gate

closure, should not be below the top of the penstock at any point, as at K. If it is, a partial vacuum will occur within the pipe, with probability of collapse of the shell. Air inlet valves to prevent vacuum, in this case, are not recommended.

The following discussion of water hammer is based on L. Allievi's theory, and several of Allievi's diagrams have been reproduced.

268. General Discussion.—Consider a cylindrical conduit or pipe which, at one end, is freely connected to a reservoir of constant level, and which delivers water at the other end through an adjustable orifice of efflux, such as a valve or turbine gate. With respect to the hydraulic gradient, therefore, the end of the pipe connected to the reservoir is the upper, and that supplied with the valve or gate, the lower end.

When the demand for water is constant, i.e., when the area of efflux is constant, the pressure at any point of the axis of pipe is also constant and is measured by the vertical distance of the pipe-axis below the hydraulic gradient, or, should friction and velocity head be neglected, by its distance below the reservoir level. If, however, the demand for water changes, and the area of the orifice is decreased or increased, a period of perturbed motion is set up in the pipe and the phenomenon called "water hammer" occurs. It is evident that a gate closure means a conversion of kinetic energy into potential energy; and a gate opening, the conversion of potential energy into kinetic energy. The liquid column is compressed or relieved of pressure; it shortens or elongates; in other words, it starts to vibrate longitudinally, and both velocity and pressure become functions not only of the static head, but also of the elastic properties of water and conduit, of the length of the pipe, of the velocity of flow destroyed or produced, and of the speed and character of gate operation.

For a closure of the orifice, the phenomenon could be compared to the behavior of a long spiral spring, having a translatory motion in an axial direction, beginning at the instant when its head spiral would hit a solid wall. The variation of velocity and also the variation of tension, which would so occur in the head spiral, would not be instantaneously transmitted to the succeeding spirals; this transmission would occur progressively, at a speed which, depending on the nature of the spring, may be very great, but never infinite. Similarly, in the pipe, the pressure rise occasioned by the closing operation at the section next to the gate will be transmitted along the pipe to the reservoir as the crest of a wave at a definite speed, and will be reflected, as a wave of depression, at the same speed, toward the gate.

In the second half of the nineteenth century, when the use of long pipe conduits became more frequent, especially for purposes of water power, a knowledge of the magnitude of increased and decreased pressures due to water hammer, in the course of operating the plants, became a necessity for correct design. Beginning about 1860, a great many attempts were made to solve the problem of water hammer, and some interesting partial or approximate solutions were obtained. Thus, in 1890, Joukowsky derived a correct formula for the maximum rise of pressure due to sudden gate closure. The first complete analytical solution of the problem was given in 1902 by L. Allievi in his "General Theory of Variable Motion of Water in Pressure Tubes." In

1913, the same author published a systematic and synthetic study of the phenomenon of water hammer, and assembled the results in easily applicable diagrams covering all possible pipe lines and methods of gate operation.

Allievi's formulae represent the laws of pressure variation in the pipe,

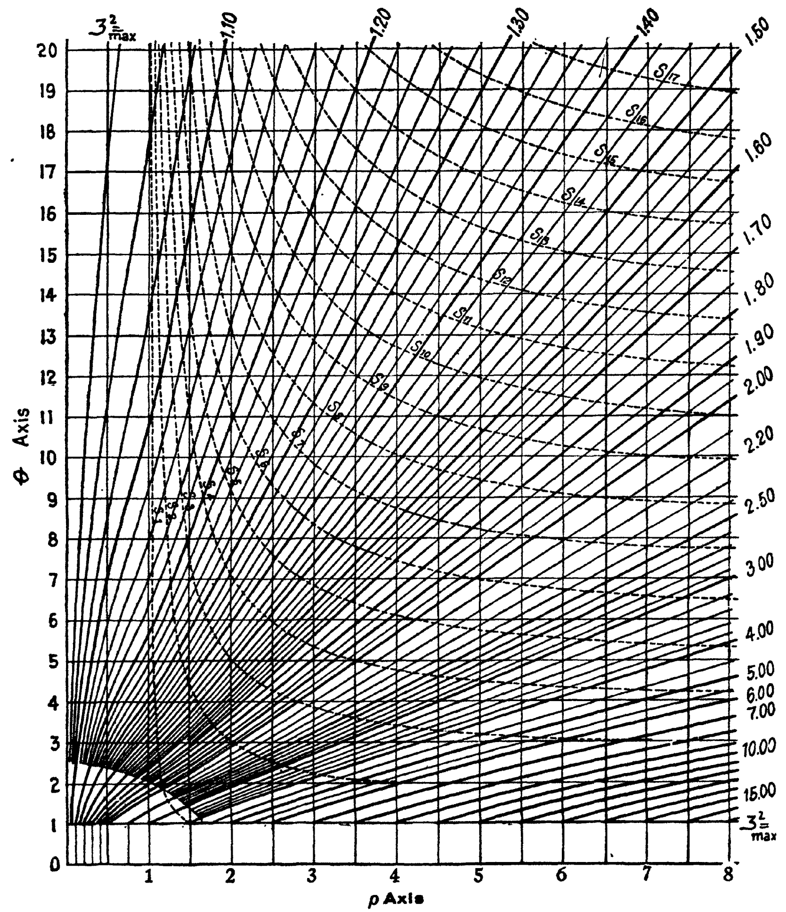

FIG. 336.—Chart for Finding Maximum Pressure of Water Hammer When Gate is Closed at Any Desired Speed. By L. Allievi.

due to the operation of the gate, as functions of two parameters, one of which, designated by the symbol " ρ," includes all constructive and functional elements of the conduit and is, therefore, called the *characteristic* of the conduit; the other parameter is the time, " θ," of the gate operation (closure or opening), measured in units of the time interval needed by the pressure wave to travel a distance twice the length of the conduit (duration of the

phase). The increase or decrease in pressure induced by water hammer, being dependent on these two parameters only, can be easily represented graphically, by using ρ and θ as coordinates.

Lack of space does not permit the authors, in this article, to set forth and discuss Allievi's mathematical theory of water hammer. However, two of his

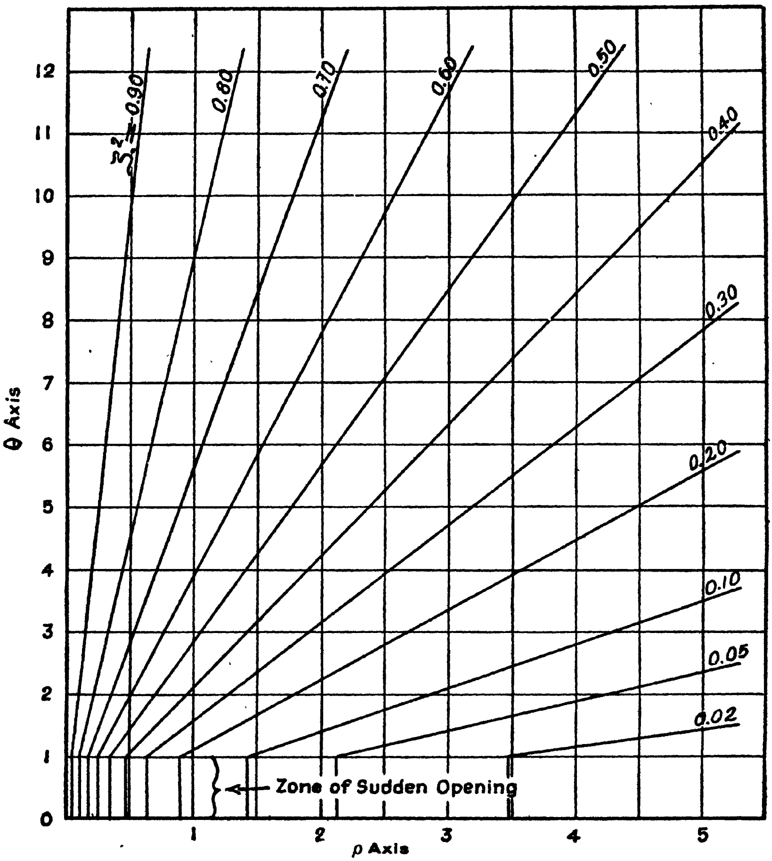

FIG. 337.—Chart for Finding the Minimum Pressure of Water Hammer when Gate is Opened from Closed Position. By L. Allievi.

most useful charts are given, and, in the following, the symbols thereon used will be defined so that the charts can be intelligently used for the solution of certain, more frequently encountered, water-hammer problems.

Of the two charts here given, the first, Fig. 336, gives the maximum pressure attained in a pipe, due to the closing of the orifice at any (uniform) speed of operation; and the second, Fig. 337, the maximum depression occurring

when the gate is opened from a closed position to any desired degree of opening and at any speed of operation.

269. The Characteristic, ρ.—

$$\rho = \frac{av}{2gy}, \quad \ldots\ldots\ldots\ldots \quad (137)$$

where g = acceleration of gravity, in feet per second, per second; = 32.2;
y = static head at lower end of pipe, in feet (of water), measured from forebay level or from elevation of water in surge tank at beginning of gate movement to tail-race level;
a = effective velocity of wave propagation in feet per second (Eq. 139);
v = the effective velocity, in feet per second, of the flow of regimen, i.e., velocity of flow before the beginning of the gate-closing operation, or, in opening the gate from a closed position, the velocity corresponding to the efflux area attained at the end of the opening operation (Eq. 138).

The effective velocity, v, is the actual velocity if the pipe is uniform in diameter. For a pipe of variable diameter in which the actual velocities, including the average velocity in the draft tube, are v_1 for length L_1 and v_2 for length L_2, etc., the effective velocity is given by the following equation:

$$v = \frac{v_1L_1 + v_2L_2, \text{ etc.}}{L}, \quad \ldots\ldots\ldots \quad (138)$$

where L is the total length of the pipe and draft tube.

If the upper end of the penstock is connected to a surge tank by a stand-pipe, the surplus or deficiency of flow in the pipe line above the tank must flow up or down the stand-pipe. Hence, for both closure and opening, the water in the stand-pipe must be suddenly started, and water hammer in the stand-pipe is created and combined with the water hammer in the penstock. Consequently, the stand-pipe should be treated as though it were a part of the penstock; its length should be included in the total length of the penstock; and the product of the velocity in the stand-pipe (corresponding to the load change) times the length of the stand-pipe should be included in Eq. (138). Should the surge tank have an inside riser as in Fig. 339, that part of such riser between the bottom of the tank and water level of regimen should be similarly treated. If the surge tank is open, the full size of the tank may be used in place of the stand-pipe; but, if the tank is large, its effect on water hammer is negligible.

The velocity of wave propagation, a, is given by the following equation, by Joukowsky:

$$a = \frac{4675}{\sqrt{1 + KB}}, \quad \ldots\ldots\ldots\ldots \quad (139)$$

where K = the ratio of the elastic moduli of water and the material of the pipe shell; and
B = the ratio of the diameter of the pipe and the thickness of shell.

Fig. 338 [1] gives values of a for steel pipes, calculated from Eq. (139), using $K = 0.01$.

For a pipe of variable diameter and thickness, a value of a should be determined for each section of constant diameter and thickness, including the stand-pipe and the draft tube, and the effective value of a determined from the following equation:

$$a = \frac{a_1L_1 + a_2L_2, \text{ etc.}}{L}, \quad \ldots \ldots \quad (140)$$

When reinforced concrete pipes are used as pressure pipes, they are so designed that the circumferential tension induced in the shell by the water

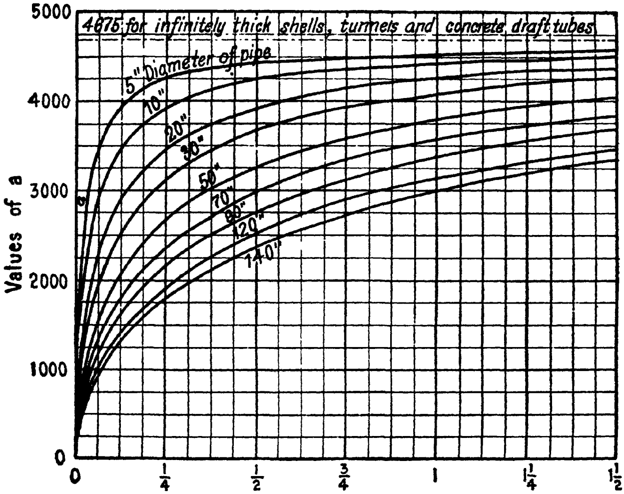

FIG. 338.—Values of a for Steel Pipe from Eq. 139.

pressure is resisted by steel reinforcement, and some tension is transmitted in the concrete. Therefore the concrete will offer additional resistance against a change in the pipe diameter, thereby making a concrete conduit actually more rigid than a steel pipe made of the same volume of steel as that of the reinforcement. Consequently, the value of a in such conduits is greater than in an equivalent steel pipe.

The writers are not aware of any experimental results that would indicate an exact method by which the factors K and B in Eq. (139) could be diminished to compensate for the composite materials used in the pipe. However, considering the elastic modulus of concrete, they propose, for safe design, to use, in the denominator of B, the equivalent thickness of a steel pipe plus one-twentieth of the thickness of the concrete shell.

[1] This diagram is not by Allievi.

The writers believe that the influence of backfilling of pipes can be neglected in the calculation of a.

In the case of a tunnel conduit in solid rock, its rigidity can be considered perfect, and a becomes 4675 ft. per second, the speed of the propagation of sound in water.

The predetermination of the magnitude of the wave velocity in a wood-stave pipe is not possible, because of the unknown influence of the change in diameter of such a pipe, under the changing pressure, due to the action of the wood staves. It has been demonstrated, however, that in many cases the wave velocity in a wood-stave pipe was a great deal lower than that of a steel pipe having the same size and strength. Values of a as low as 250 ft. per second for wood-stave pipe have been found by experiment.[2]

It may be interesting to mention that the characteristic ρ can be shown to be equal to one-half of the square root of the ratio of the kinetic energy and potential energy contained, during regimen, in a unit length of the conduit. The characteristic has a small value for high heads and a large value for low heads.

270. The Time, θ.—

$$\theta = \frac{at}{2L} = \frac{t}{\mu}. \qquad \qquad (141)$$

where t = time in seconds of gate operation, assumed to be uniform; in the case of closure, this time is the time necessary to close the gate completely, and in the case of an opening operation for placing the conduit in service, it is the time in which the gate is opened to the extent which will produce the desired velocity of flow. (See explanation, Sec. 271.)

L = length of conduit in feet, from the tail race (including draft tube) to the forebay; or, if a surge tank is used, from the tail race to water surface in the stand-pipe of the surge tank.

$\mu = \dfrac{2L}{a}$ = duration of the phase, unit of time for measuring θ.

271. Governor Movement.—Manufacturers of turbine governors find that the gate travel in the central portion of the stroke is more rapid than the average rate of speed of the complete stroke, owing to relatively slow motion at the start and finish of the stroke. This feature has no effect in the case of gate opening, as it is only the total time of movement that affects water hammer.

For gate closure, however, the maximum rate of gate movement has an important bearing on the amount of water hammer. To simplify the method, it will be conservative to assume a reduced value of t corresponding to the time of gate operation if the gate moved throughout the stroke at a constant rate equal to the maximum rate.

The I. P. Morris Department,[3] for instance, recommends using 85 per cent

[2] See experiments by Marks, Wing and Hoskins. Trans. Am. Soc. C. E., Vol. XL, p. 489.

[3] William Cramp and Sons.

of the nominal value of t for gate closure, conforming to the ratio of the average rate of speed of its governors to the rate of speed at the critical portion of the stroke. It also recommends using 100 per cent of the nominal speed for gate opening. However, several recent tests have indicated that 85 per cent for gate closure is not sufficiently conservative. The authors recommend using 50 per cent of the nominal governor time for gate closure for all governors having a closing time less than about eight seconds. For very slow governors, this factor may be increased, because the slower speed at the start and finish of the stroke occurs for a relatively shorter time. Experiments are badly needed.

272. Critical Governor Time.—The expression $\mu = \frac{2L}{a}$ is the time, in seconds, required for the water-hammer wave to travel from the gate to the upper end of the pipe and return. This is called the "critical time" because the laws of water hammer for gate movement in less than critical time are entirely different from those for gate movement in more than critical time. Values of t close to the critical time are very seldom used in practice: and, as a matter of fact, such fast gate movement should not be used unless the designer has more knowledge of the theory of water hammer than it has been possible to give here. The uncertain feature of such quick movement is the values of water hammer at other points than at the turbine.

273. The Symbol ξ.—The symbol ξ is a measure of the water hammer and is obtained from Figs. 336 and 337 after ρ and θ are known. Then

$$\xi^2 = \frac{y + h}{y}, \qquad \qquad (142)$$

where h is the pressure of water hammer in feet (of water) and y is the static head at the lower end of the pipe, in feet, measured as indicated in Sec. 269. Therefore, ξ^2 is the ratio of the total pressure, including water hammer, to the static head.

In the case of a surge tank, y is measured to water surface in the riser pipe of the tank, and the calculated water hammer, h, should be increased by the change of water level, h_R, in the surge tank (or riser pipe) which will occur during the period in which the water hammer is active. For a gate closure the water hammer continues to oscillate for a considerable period of time, and for this condition h_R should be the maximum rise in the tank. For gate opening, however, the water-hammer waves die out quickly, and it seems reasonable to add to the calculated negative water hammer only the drop of water surface in the surge tank which would occur during the gate movement. Therefore, from Eq. (142), the total water hammer is,

$$h_T = h + h_R. \qquad \qquad (143)$$

274. The Curves s.—The curves s in the chart, Fig. 336, indicate the time, in terms of units μ, which elapses from the beginning of closure to the instant of the occurrence of the maximum pressure. These curves will show whether or not a partial closure can produce as great a water hammer as would occur

if the gate operation continued to complete closure. For the case of a partial closure, t, in $\theta = \frac{t}{\mu}$, must be taken as the time necessary to close the gate completely, at the same rate of speed as was assumed for the partial closure. If, then, the subscript of the corresponding curve s should indicate a shorter time for the occurrence of the maximum pressure than the actual time of gate movement in terms of μ, the same maximum pressure would be attained as if the gate were completely closed. Should the actual time of gate movement be less than the subscript of the corresponding curve s, the maximum

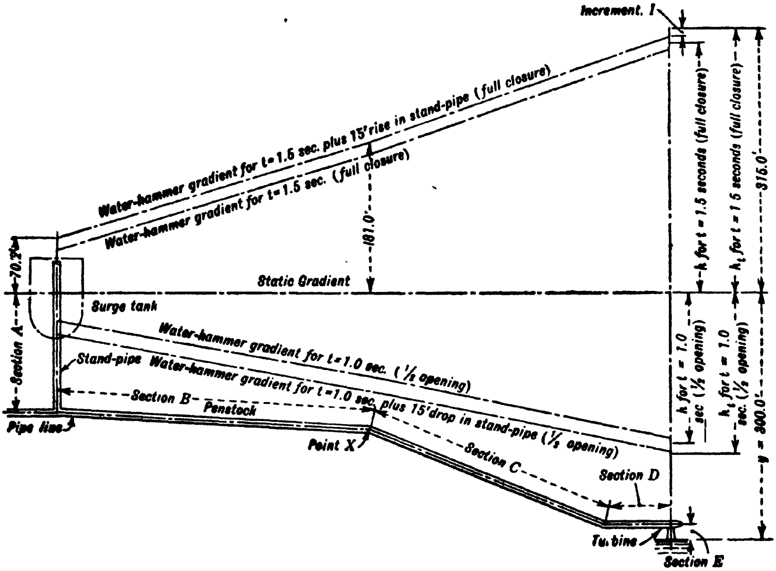

FIG. 339.

pressure found on the chart could not be attained by this particular partial closure.

275. Pressure Conditions along the Pipe.—The pressures at points along the pipe are also functions of time-pressure relations which are not treated in this article. However, it is sufficiently exact to assume for design that, for a governor time greater than one interval $\left(t > \frac{2L}{a}\right)^*$, the magnitude of the maximum superpressure or depression found at the gate diminishes from the gate to the point of open water and has a value at any point, n, equal to that given in the following equation:

$$h_n = h\left(\frac{v_1L_1 + v_2L_2 + \ldots v_nL_n}{vL}\right). \quad \ldots \quad (144)$$

* "The critical time," see Sec. 272.

where h_n = the water hammer at point n;
h = the water hammer at the turbine;
v = the effective velocity for the whole pipe from Eq. (138);
L = the total length of the pipe; and
L_n = the length of the pipe from point of open water to point n.

276. Numerical Examples.—To explain the application of the foregoing principles, assume the following conditions for the penstock shown in Fig. 339.

TABLE LIV

PENSTOCK CHARACTERISTICS

SECTION	*A*	*B*	*C*	*D*	*E*
Diameter, ft.	7.0	8.0	7.5	7.0	9.0
Length, ft.	145	380	300	80	20
Thickness, in.	$\frac{3}{8}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{3}{4}$	Concrete
"*a*" from Fig. 338	2550	2450	2750	3200	4675
Area, sq. ft.	38.5	50.3	44.2	38.5	63.6
Velocity for Q of 487 sec.-ft.	12.67	9.68	11.02	12.67	7.65

$y = 300$;
$L = 925$.

From Eq. (138),

$$v = \frac{12.67\times145+9.68\times380+11.02\times300+12.67\times80+7.65\times20}{925} = 10.8.$$

From Eq. (140),

$$a = \frac{2550\times145+2450\times380+2750\times300+3200\times80+4675\times20}{925} = 2680.$$

Duration of phase, from Eq. (141),

$$\mu = \frac{2L}{a} = \frac{2\times925}{2680} = 0.69 \text{ sec.} = \text{" critical time."}$$

Assume h_R for Eq. (143) to be 15 ft. for both opening and closing.

Gate closure.—(*a*) To find the minimum closing time for the condition that the maximum pressure increase above static at the turbine should not be more than 65 per cent.

Then,

$$h_T = 0.65y = 0.65\times300 = 195.$$

From Eq. (143),

$$h = 195 - 15 = 180.$$

From Eq. (142),

$$\xi^2 = \frac{300+180}{300} = 1.60.$$

From Eq. (137),

$$\rho = \frac{2680\times10.8}{2\times32.2\times300} = 1.50.$$

From Fig. 336 the ordinate of a point on the curve $\xi^2 = 1.60$ with abscissae $\rho = 1.50$ is,

$$\theta = 3.2.$$

From Eq. (141),

$$t = \frac{2 \times 925 \times 3.2}{2680} = 2.21 \text{ sec. actual governor time.}^4$$

The position of the point determined by the coordinates $\rho = 1.5$ and $\theta = 3.2$ indicates, by the subscripts of the curves s, that the specified maximum pressure occurs at 1.6 intervals or $1.6\mu = 1.6 \times 0.69 = 1.1$ sec. after the beginning of gate operation. A partial closure would result in the same pressure if the gate movement had proceeded for only this length of time and then had stopped. In other words, the water-hammer pressure after the time 1.1 sec. is less than at that moment.

(b) To find the maximum pressure increase at the turbine, if the actual (not nominal)[4] governor closing time be reduced to 1.5 sec.

From Eq. (141),

$$\theta = \frac{2680 \times 1.5}{2 \times 925} = 2.17.$$

And as before,

$$\rho = 1.5.$$

These in Fig. 336 give,

$$\xi^2 = 2.0.$$

Therefore, the pressure increase is, from Eq. (142),

$$2 = \frac{300 + h}{300},$$

or

$$h = 300 \text{ ft. above static.}$$

And from Eq. (143) the total rise is

$$h_T = 300 + 15 = 315 \text{ ft. above static.}$$

(c) For example (b), find the pressure in the penstock at point x, Fig. 339.

Eq. (144), applied to Sections A and B, above x, gives,

$$h_n = 300\left(\frac{12.67 \times 145 + 9.68 \times 380}{10.8 \times 925}\right) = 166 \text{ ft. above static.}$$

And the total rise at point x is, from Eq. (143),

$$h_{n_T} = h_n + h_B = 166 + 15 = 181 \text{ ft. above static.}$$

This rise is plotted in Fig. 339 together with the corresponding rise at other points, giving the maximum water-hammer gradient for the penstock for a 1.5-sec. closure. $h_T = 315.0'$, as previously calculated, is the water-hammer induced by the stoppage of all the flow including that in the draft

[4] This is to be increased by the required percentage to obtain nominal governor time. See definition of t in Sec. 270.

tube. Also h_T minus the increment I (Fig. 339) is that produced by the stoppage of all the flow above the turbine calculated as described for point x. The increment I at the turbines, therefore, is the water hammer in the draft tube, which causes added stress on the turbine gates only. The water-hammer gradients at the stand-pipe have reference to the foot of the stand-pipe, the pressure diminishing linearly up the stand-pipe to zero at water surface.

Gate opening.—(*a*) The penstock is to be placed in service by opening the gate to produce 243.5 sec.-ft. (one-half opening) in 1.0 sec. What is the resulting decrease in pressure at the turbine?

One-half the discharge corresponds to one-half the effective velocity previously determined, or

$$v = 0.5 \times 10.8 = 5.4.$$

From Eq. (141),

$$\theta = \frac{2680 \times 1}{2 \times 925} = 1.45.$$

From Eq. (137),

$$\rho = \frac{2680 \times 5.4}{2 \times 32.2 \times 300} = 0.75.$$

These in Fig. 337 give,

$$\xi^2 = 0.39.$$

Therefore, the pressure decrease is, from Eq. (142),

$$h = 300 \times 0.39 - 300 = -183 \text{ ft. below static}$$

And, from Eq. (143), the total drop is,

$$h_T = -(183 + 15) = -198 \text{ ft. below static.}$$

(*b*) For the preceding example, find the resulting decrease in pressure at other points on the penstock.

These values have been computed from Eq. (144), as previously shown for gate closure, and plotted on Fig. 337.

277. Pressure Conditions after Closure.—At the instant that the gate is completely closed, and thereafter, the pressure at the turbine (and proportionally at other points on the penstock) will oscillate between the limits.

$$y + h_1 + h_R \quad \text{and} \quad y - h_1 + h_R,$$

where h_1 is the increase in pressure at the instant of closure, and is not necessarily the maximum found from Fig. 336, because, as previously explained, the maximum increase in pressure may occur before complete closure. The exact determination of h_1 at the end of complete closure can only be done by analyzing the time-pressure relations of water hammer, which is outside the scope of this chapter. The use of the maximum pressure, h, is therefore recommended as being on the safe side.

278. Pressure Conditions after Opening.—It can be proved that an opening operation does not induce dangerous counterblows of superpressure.

In fact, the superpressure at the turbine cannot exceed 23 per cent of the static head.

279. Water Hammer in the Pipe Line.—All that has been said heretofore pertains strictly to the penstock, i.e., between the surge tank and the turbines. For determining the water hammer in the pipe line, i.e., between the forebay and the surge tank, the following method should be used.

Equation (144), with Eq. (143), gives a method of determining water hammer at any point in the penstock, as explained in Example *c* of Sec. 276. In this example the water hammer at the base of the riser pipe is 70.2 ft. as shown in Fig. 339. This amount of water-hammer wave travels undiminished up the pipe line to a point determined by the following equation, then reduces linearly to zero at the forebay.

$$l_P = \frac{ta_P}{2}, \qquad \qquad (145)$$

where l_P = the distance, in feet, from the forebay to the point in question;
a_P = the effective velocity of the wave propagation in the pipe line, in feet per second;
t = time, in seconds, of gate operation as used for computing the penstock water hammer.

If l_P from Eq. (145) results in a quantity greater than the length of the pipe line, the water hammer may be taken as equal to that computed at the base of the riser, diminishing linearly to zero at the forebay.[5]

In the aforementioned example, let $a_P = 2000$. Therefore, as t, in the example is 1.5, we have from Eq. (145).

$$l_P = \frac{1.5 \times 2000}{2} = 1500 \text{ ft. from the forebay.}$$

Eq. (145) may also be used to determine the distance the maximum surge-tank surge will travel up the pipe line, using for t the time occupied by the surge.[6]

280. Factors of Safety.—The allowance which should be made to compensate for possible inaccuracies in computed water hammer depends upon the character of the installation. Allievi's theory of water hammer is correct, but the curves and equations given in this chapter, which are based on Allievi's theory, are correct for the following conditions only:

(*a*) Diameter and thickness of the pipe is constant;
(*b*) The gate-closure time, t, is uniform;
(*c*) The upper end of the pipe is at open water.

For more complicated conditions, the application of Allievi's theory is very complicated and not within the scope of this chapter. Consequently, for such conditions, approximations are made to provide a practical method of

[5] Actually, if the computed l_P is greater than the length of the pipe line, the water hammer, both in the pipe line and in the penstock, will be less than shown by the foregoing methods; but this is a very unusual case for a very short pipe line, and the exact determination of the water hammer for such cases is very difficult to make.

computing water hammer, and due allowance must be made for possible errors in such approximations.

It is therefore recommended that the computed water hammer, h (not h_T of Eq. (143)), both positive and negative, be multiplied by the following factors x to allow for possible inaccuracies in its computation.[6]

(1) Penstock of constant diameter and approximately constant thickness. Upper end at open forebay................ $x = 1.00$

(2) Penstock of variable diameter and variable thickness. Upper end at open forebay......................... $x = 1.10$

(3) Penstock of variable diameter and thickness below a surge tank... $x = 1.20$

(4) Pipe line between forebay and surge tank of constant diameter and approximately constant thickness............. $x = 1.30$

(5) Pipe line between forebay and surge tank of variable diameter and variable thickness......................... $x = 1.50$

281. Bibliography.—

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2. General Theory of Perturbed Flow of Water in Pressure Conduits, by Allievi. Revue Mécanique, 1903.
3. Theory of Water Hammer, by Allievi. Revue Mécanique, 1913.
4. Water Hammer, by Escher. Die Turbine, 1910.
5. Pulsations in Pipe Lines, by Vensano. Trans. Am. Soc. C. E., Vol. LXXXII, 1918.
6. Pressures in Penstocks, by Gibson. Trans. Am. Soc. C. E., Vol. LXXXIII, 1920.
7. Fall in Pressure in Penstocks, by Kerr. N. E. L. A. Convention, 1924.

[6] EDITOR'S NOTE.—If this method is followed, the factor of safety used in adopting the working stresses for the pipe should not allow for inaccuracies in the loading, as explained in Sec. 184.

CHAPTER XXV

SURGE TANKS

BY WILLIAM P. CREAGER

282. Theory.—Water-hammer pressure, the theory of which is treated in Chapter XXIV, is created in long closed conduits by sudden closure of the turbine gates. The water-hammer pressure provides the necessary force to retard the flow in the conduit when load is rejected by the turbine. For very long conduits, the water hammer corresponding to normal operation of the turbine may be very great and may require extraordinary strength of the conduit to withstand it; and the violent fluctuations of pressure in the conduit may seriously interfere with proper turbine regulation. For sudden opening of the gates, the resulting negative water hammer, or reduction of pressure,

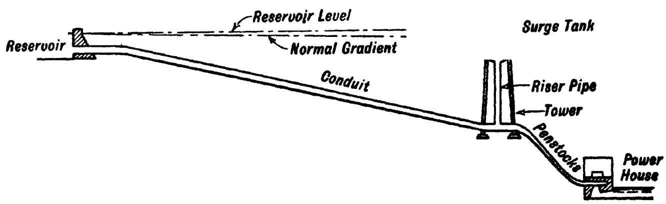


FIG. 340.—Usual Location of Surge Tank.

provides the necessary force to accelerate the water and is correspondingly objectionable for very long conduits on account of difficult turbine regulation.

The simplest means of eliminating positive water-hammer pressure is to provide a by-pass to take the rejected flow. This may be accomplished by installing a relief valve [1] at the turbine or, as shown in Fig. 340, a surge tank at the lower end of the conduit. The relief valve is very effective for gate closure in that it provides a place for the surplus water to go; but for gate opening, it naturally cannot supply the necessary water demanded by the turbine, and consequently it has no effect on negative water hammer. For these reasons, a surge tank is invariably provided at the lower end of all very long closed conduits.

In the case of a simple surge tank, shown in Sketches *A* and *B* of Fig. 341, the water simply flows into the tank when rejected by the turbine. As the

[1] Described in Sec. 337. (Pressure Regulators in Turbines.)

water rises in the tank, a retarding head is created, which decreases the conduit velocity. When the velocity in the conduit is reduced to that demanded by the turbine, the water in the tank starts to fall and fluctuates up and down like the swing of a pendulum until dampened out by friction. For a sudden increase in load, the additional discharge required by the turbine flows out of the tank, and the consequent lowering of water surface creates an accelerating head which increases the flow in the conduit. When the conduit discharge corresponds to the turbine demand, the water surface in the tank ceases to fall. The action of a simple surge tank after a sudden opening of the turbine gates is indicated in Fig. 342.[3]

Ordinarily, the tank is so designed that the water will not spill over the top under the most drastic condition of load rejection. However, in special cases of low-head installations, where conditions are favorable, the tank is allowed to spill over; but it is usually found more economical to provide a sufficiently large tank than to make the frequently expensive provision to take care of the overflow with safety.

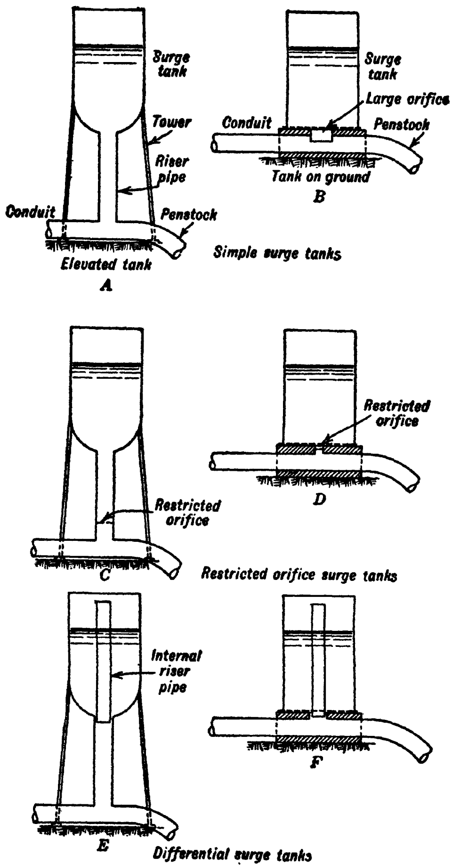


FIG. 341.—Types of Surge Tanks.

283. The Simple Surge Tank.—Sketches A and B of Fig. 341 show outlines of "Simple" surge tanks. The surge tank is always located as close as possible to the power house, in order to reduce the length of penstock to a minimum, and preferably on high ground, to reduce the height of the tower. If the site of the tank is sufficiently high, the tower shown in Sketch A is omitted, the tank rests directly on a concrete foundation containing the conduit, and an orifice in the top of the conduit is provided, of ample size to permit flow in and out of the tank.

The conduit accelerating and retarding heads, induced by a change of water surface in the simple tank, accumulate slowly, corresponding to the

[3] Computed by Roy Taylor. See Trans. Am. Soc. C. E., Vol. LXXVIII, p. 789.

gradual change of water level in the tank. The action of the simple tank is sluggish as compared with that of other types, and, as will be shown later, the simple tank requires the greatest volume. Hence, except for special cases, it is the most expensive and is seldom adopted in preference to other types.

284. The Restricted-orifice Surge Tank.[1]—Sketches *C* and *D* of Fig. 341 show outlines of "Restricted-orifice" surge tanks. The distinctive feature of this type of tank is a restricted orifice, installed between the conduit and the tank. The object of the restricted orifice is to create an appreciable friction loss when the water is flowing to or from the tank.

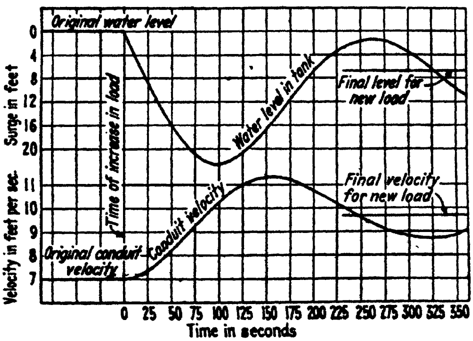


Fig. 342.—Action of a Simple Surge Tank.*

*From Roy Taylor in Trans. Am. Soc. C. E., Vol. LXXVIII, p. 789.

When load is rejected by the turbine, the surplus water passes through the restricted orifice, and immediately a retarding head, equal to the loss due to the restricted orifice, is built up in the conduit. The size of the orifice may be designed for any desired retarding head. If the orifice is of large size, the tank passes into the "simple" tank class and the retarding head is negligible. If the orifice is infinitely small, the retarding head is equal to the water hammer in the conduit with no surge tank. The size adopted is usually such that the initial retarding head, for full load rejected by the turbine, is approximately equal to the ultimate rise of water surface in the tank.

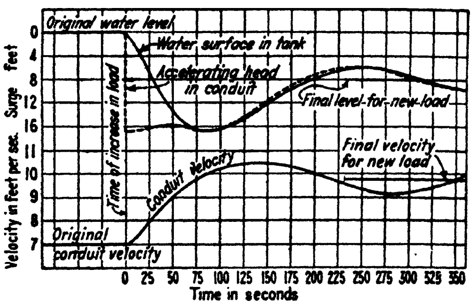


Fig. 343.—Approximate Action of a Restructed-Orifice Surge Tank.

The more quickly the accelerating and retarding heads are applied, the more effective will be the surge tank in the adjustment of the conduit discharge, and hence less water will have to be stored in or delivered from the tank. The tank may therefore be smaller.

Figure 343 shows the action of a restricted-orifice tank for an increase in load, and Fig. 345 shows a comparison of the restricted-orifice tank, and the simple tank. Fig. 343 shows that, when the turbine gates are opened, an

[1] See footnote under sec. 285.

accelerating head in the conduit is immediately created. The water in the tank gradually lowers and, at the end of the first quarter cycle (85 seconds), the pressure in the conduit corresponds to the level in the tank, and the flow out of the tank has been reduced to zero.

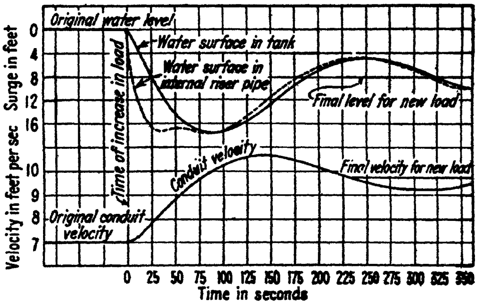


FIG. 344.—Action of a Differential Surge Tank.*

* Roy Taylor in Trans. Am Soc. C. E, Vol. LXXVIII, p. 789.

The restricted-orifice tank offers greater efficiency and economy than the simple tank; but the desirable sudden creation of accelerating and retarding head in the conduit also induces correspondingly sudden fluctuations of head on the turbine, which the governors may have difficulty in accommodating. The simple tank is ideal, as far as ease of governing is concerned, in that the head changes are so gradual that even a very slow-acting governor has no difficulty in following the change of pressure. On account of the sudden pressure changes in restricted-orifice tanks, this type cannot be adopted for many installations where close governing is required and where the cost of the necessary additional inertia of the rotating elements of the generating units, to compensate for such fluctuations, would be prohibitive.

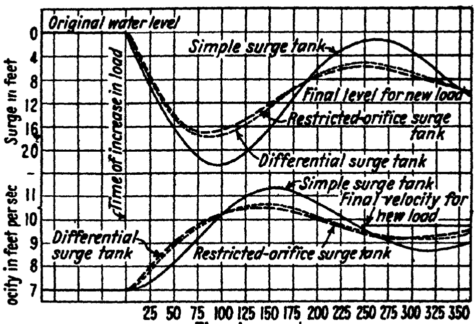


FIG. 345.—Comparison of Action of Different Surge Tank Types.

285. The Differential Surge Tank.—Sketches *E* and *F* of Fig. 341 show outlines of the Differential surge tank.[4] This type is designed to provide a compromise between the simple tank and the restricted-orifice tank. It differs from the simple tank only in having the additional internal riser, as shown. The internal riser is of smaller diameter than the connection to the conduit. At the base of the internal riser there is an annular port, communicating with the tank. The area of this port is proportioned to suit the conditions under which the tank is to operate. The characteristics of the surge depend upon the area of the port.

[4] Since this text has been written, the writer has been advised that the "Restricted-orifice Tank" and the "Differential Tank" are both patented articles and that the term "differential" is a coined word which applies to both types.

In the differential tank, operating for gate opening, the water first falls in the internal riser, establishing in a short time a relatively large accelerating head on the conduit. The level of the tank falls slowly, supplying the demanded increment of flow through the ports at the base of the riser. When the gates are closed, water rises in the internal riser, establishing a retarding head on the conduit as well as a differential head on the port, which forces the water rejected by the turbines through the port into the tank.

The action of the differential tank for an increase in load is shown in Fig. 344; and in Fig. 345 is shown a comparison of the action of this type of tank with that of a simple and a restricted-orifice tank.

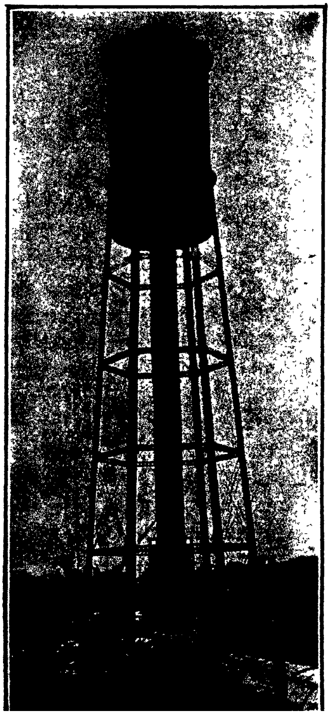


FIG. 346.—Forty-foot Differential Tank 245 Feet High at Browns Falls Plant of Northern New York Utilities, Inc.

A comparison of Figs. 343 and 344 shows that the action of the differential tank is similar to that of the restricted-orifice tank, except that the initial pressure change and head on the turbine, instead of occurring instantly as in the restricted-orifice tank, or very slowly as in the case of the simple tank, occurs quickly enough for good efficiency of the tank and is still spread over a period long enough to enable the governors to adjust the turbine gates to compensate for the change in head.

286. Design of Surge Tanks.—The theory of regulation of conduit flow by the use of surge tanks is quite involved, and is, in fact, too intricate for proper treatment within the scope of this book. Correct design is so important to successful operation that the data should be very carefully analyzed by an engineer who has devoted much time to the study of the theoretical considerations and who has had the rare opportunity of applying the theory to many practical cases.

However, simple methods have been devised, by which the approximate dimensions of a surge tank can be obtained for purposes of preliminary estimates, economy studies, and other obvious uses. Such methods should never be used for final designs.

Figure 347 is a diagram made by R. D. Johnson for the approximate solution of surge-tank problems. The diagram is made for both differential and simple tanks. Mr. Johnson states that the curves for the simple tank are not extremely accurate and recent investigations indicate that some, or possibly all of them, should be slightly raised for the larger values of K'_a. It is

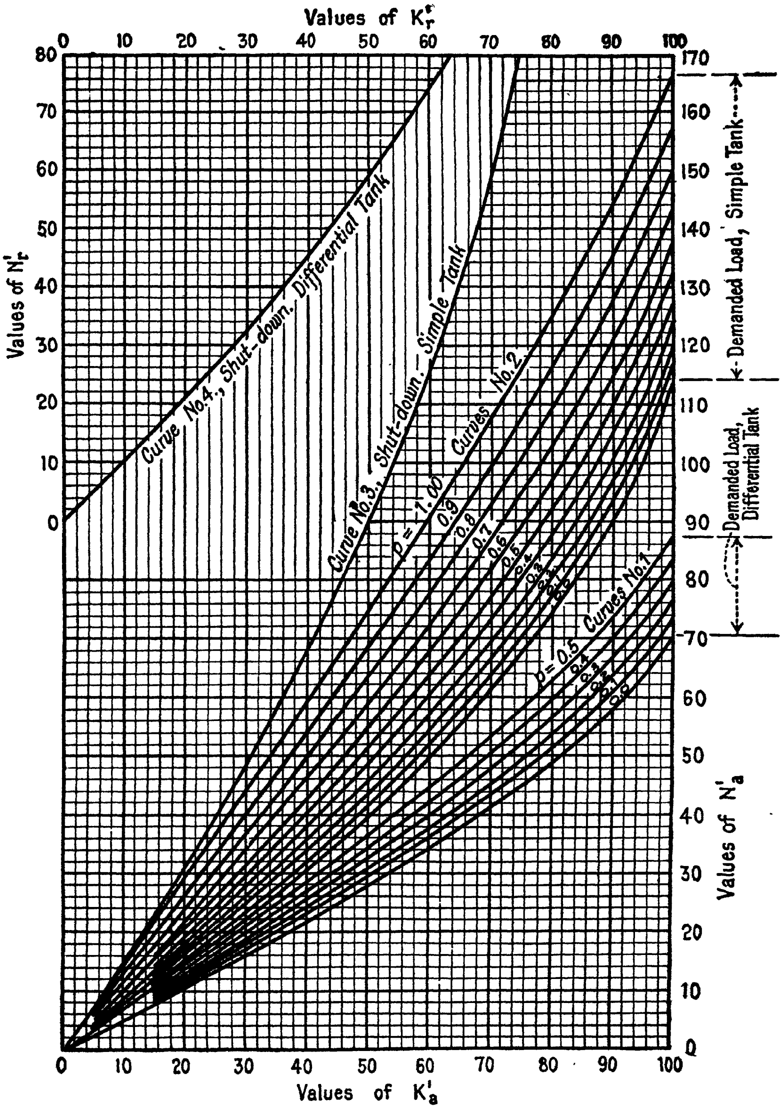


FIG. 347.—Johnson Surge Tank Diagram.

likely, for example, that the top curve would pass through the point whose co-ordinates are 141.5 and 87.0 instead of 141.5 and 89.5. The diagram is used in the following manner:

In the following nomenclature, the unit of measure is the foot.

Let A = area of the conduit;
R = area of the internal riser of a differential surge tank;
L = length of the conduit from forebay to surge tank;
F = net area of surge tank, i.e., area in excess of internal riser area;
f_a = total loss of head in conduit, including velocity head at surge-tank riser, assumed for accelerating conditions;
f_r = same for retarding conditions;
H = net head on the turbines, used for test of incipient stability;
d_a = lowest draft on pond, assumed for accelerating conditions;
d_r = highest flood height on crest, assumed for retarding conditions;
y_a = fall of water level in the tank from its initial position previous to a load increase;
y_r = rise of water level in the tank from its initial position previous to a load decrease;
Y_a = maximum fall of water level in the tank below crest of dam;
Y_r = maximum rise of water level in the tank above crest of dam;
c = friction characteristic of the conduit, or $cv^2 = f$;
K'_a and N'_a = constants for accelerating conditions;
K'_r and N'_r = constants for retarding conditions;
v_1 = initial conduit velocity before acceleration begins;
v_2 = initial conduit velocity before retardation begins;
v_c = critical conduit velocity;
g = acceleration of gravity = 32.2;
p = percentage of velocity change;

Since friction in the conduit decreases the upward surges for retarding conduit-velocity conditions, and increases the downward surges for accelerating conditions, the value of c to be adopted should correspond to the minimum conceivable friction for retarding conditions and the maximum conceivable friction for accelerating conditions. Possible variations in friction for various types of conduits are given in Table XXIII of Sec. 73.

In determining the maximum discharge of the turbines, it should be remembered that the guarantees of the manufacturers are sometimes exceeded. Therefore, a margin of safety should be applied when computing the maximum conduit velocity.

It is customary to design the surge tank to accommodate full load rejected by the turbines, such as might occur after a short circuit on the transmission line. The choice of the load increase for which the tank should be designed depends upon the number of turbines in the plant, the nature of the power market, and other considerations. It is possible for the operators to throw full load on the plant suddenly; but this is quite unlikely to be a normal operating condition, particularly if more than one turbine is installed. It is customary to adopt a load change corresponding to that which may be expected

from normal power-market demands. On large systems, the sudden changes of load demand are insignificant; while for an individual plant carrying a wildly fluctuating load, the percentage load increase may be quite severe. For plants of more than one unit, supplying fairly large systems, an increase of load from three-quarters to full is a frequent assumption.

The various steps necessary for the use of the diagram, and an explanatory example, will now be given.

ACCELERATION

Step 1. Compute the minimum possible full gate discharge, Q, of the turbines and the corresponding v_2 in the conduit under lowest head-water conditions and the value of f_a in the next step.

Step 2. Compute the maximum possible friction loss, f_a, in the conduit, for the computed value of v_2 in the first step. Obviously, these two steps are interdependent and must be solved by trial.

Step 3. Compute c from

$$c = \frac{f_a}{v_2^2}. \qquad (146)$$

Step 4. Compute F in terms of N'_a from

$$F = \left(\frac{N'_a}{100cv_2\sqrt{\frac{2g}{AL}}}\right)^2. \qquad (147)$$

Step 5. Assume a velocity change from v_1 to v_2 for which the surge tank is to be designed.

Step 6. Compute K'_a in terms of y_a from

$$K'_a = \frac{100c(v_2^2 - v_1^2)}{y_a}. \qquad (148)$$

Step 7. Assume several values of surge, y_a, as indicated in Table LV.

Step 8. For each surge, compute the corresponding K'_a from Eq. (148) of Step 6.

Step 9. Compute p from

$$p = \frac{v_2 - v_1}{v_2}. \qquad (149)$$

Step 10. With K'_a and p from Steps 8 and 9, find the corresponding N'_a from Curve No. 1 or No. 2 of the diagram of Fig. 347.

Step 11. With this value of N'_a, compute the corresponding value of F from Eq. (147) of Step 4.

Step 12. Compute the departure of the original friction gradient from crest level, from

$$\text{Departure} = cv_1^2 + d_a.$$

Step 13. This departure, added to the assumed surge, y_a, in Step 7, gives the lowest surge, Y_a, below crest level.

Step 14. Compute, from the value of F derived in Step 11, the corresponding tank diameter from

$$D = \sqrt{\frac{F + R}{0.785}}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (150)$$

The diameter, R, of the internal riser may be assumed to be the same as that of the conduit, which is an approximation on the safe side, as the riser is usually slightly smaller than the conduit.

Step 15. Values of Y_a and D from Steps 13 and 14, as shown in Table LV, may now be plotted in the form of a curve as in Fig. 348.

RETARDATION

Step 1. Compute the maximum possible full gate discharge, Q, of the turbines, and the corresponding velocity, v_2, in the conduit, under highest head-water conditions, and the value of f_r in the next step. Note that this value of Q is larger than that for acceleration because the original net head is greater.

Step 2. Compute the minimum possible friction loss, f_r, in the conduit for the computed value of v_2 in the first step.

Step 3. Compute c from

$$c = \frac{f_r}{v_2^2}.$$

Step 4. Compute N'_r in terms of F, from

$$N'_r = \left(100cv_2\sqrt{\frac{2g}{AL}}\right)\sqrt{F}. \quad . \quad . \quad . \quad . \quad . \quad . \quad (151)$$

Step 5. Compute y_r in terms of K'_r, from

$$y_r = \frac{100cv_2^2}{K'_r}. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (152)$$

Step 6. Assume several values of surge-tank area, F, as indicated in Table LVI.

Step 7. For each area, compute the corresponding N'_r from Eq. (151).

Step 8. With this value of N'_r, find the corresponding value of K'_r from Curve No. 3 or 4 of the diagram of Fig. 347.

Step 9. With this value of K'_r, find the corresponding value of y_r from Eq. (152).

Step 10. Compute the departure of the original friction gradient from crest level, from

$$\text{Departure} = d_r - cv_2^2.$$

Step 11. This departure, added algebraically to the value of y_r from Step 9, gives the highest surge, Y_r, above crest level.

Step 12. Compute, from the assumed values of F in Step 6, the corresponding tank diameter from Eq. (150).

Step 13. Values of Y_r and D from Steps 11 and 12, as shown in Table LVI, may now be plotted in the form of a curve as in Fig. 348.

Ordinates between the two curves of Fig. 348 show the required height of the tank for any adopted value of diameter. However, the curves should now be tested for "incipient stability" and "critical velocity."

TEST FOR INCIPIENT STABILITY

To insure incipient stability, that is, steady conditions under slight load changes, D. T. Thoma [5] has shown that the area of the tank should not be less than

$$F_M = \frac{AL}{2gcH}. \qquad (153)$$

Or, in terms of diameter, the diameter should not be less than,

$$D_M = \sqrt{\frac{R + \dfrac{AL}{2gcH}}{0.785}}. \qquad (154)$$

The value of c adopted for retardation and the smallest possible net head, H, under full-load conditions, should be used.

Equations 153 and 154 are theoretical expressions embodying the assumption of constant efficiency of turbine. R. D. Johnson has shown [6] that, for operation on the drooping side of the efficiency curve, the tank dimensions used must be still larger. However, as the shape of the efficiency curve is not usually known for preliminary investigations, the approximate diameter of the tank, to insure incipient stability, may be assumed 25 per cent larger for the differential tank and 40 per cent larger for the simple tank.

TEST FOR CRITICAL VELOCITY

For a given area of tank, there is a critical velocity which, if reduced to zero, will give the maximum height of surge. R. D. Johnson's equation for the critical velocity is

$$v_c = \frac{0.1}{c}\sqrt{\frac{AL}{F}}. \qquad (155)$$

Or, in terms of diameter,

$$v_c = \frac{0.1}{c}\sqrt{\frac{AL}{0.785D^2 - R}}. \qquad (156)$$

The value of c that was adopted for retardation should be used.

The critical velocity may be greater or less than the full-load velocity, v_2. If the critical velocity is greater than v_2, no correction need be made in

[5] Zur Theorie des Wasserschlosses bei Selbsttätig Geregelten Turbinenanlagen, by Dr. Ing., D. T. Thoma.

[6] Trans. Am. Soc. C. E., Vol. LXXXII, p. 269.

the curves of Fig. 348; except that, if the critical velocity is only slightly greater than v_2, it might be thought advisable to design for the critical velocity, than which no velocity could give a greater surge. If the critical velocity is less than v_2, it should be used for determining a revised surge to fix the height of the tank of the given size.

THE ADOPTED SIZE OF TANK

From Fig. 348 it is seen that the diameter varies inversely as the height. Consequently, any combination of diameter and height that will correspond to the curves of Fig. 348 will fulfil the requirements. The combination adopted should be such as to make the cost of the tank a minimum, it being borne in mind that the tank should be as large in diameter as possible, consistent with economy, to limit the extent of the surges for which the governor must compensate.

287. Explanatory Example.—The following example, for the approximate design of a differential surge tank, is explanatory of the foregoing method.

ACCELERATION

Step 1. Let $Q = 292$ sec.-ft.

$v_2 = 6.60$ ft. per sec.

Step 2. Let $h_f = 18.4$ ft.

Step 3. Compute $c = \dfrac{18.4}{6.6^2} = 0.422.$

Setp 4. Let $A = 44.18$ sq. ft.

Let $L = 12{,}800$ ft.

Compute
$$F = \left(\frac{N'_a}{100 \times 0.422 \times 6.6\sqrt{\dfrac{2 \times 32.2}{44.18 \times 12{,}800}}}\right)^2,$$

or
$$F = \left(\frac{N'_a}{2.98}\right)^2.$$

Step 5. Let Q change from 219 to 292.
Corresponding velocity change is

$v_2 = 6.6,$

$v_1 = 4.95.$

Step 6. Compute
$$K'_a = \frac{100 \times 0.422(6.6^2 - 4.95^2)}{y_a},$$

or
$$K'_a = \frac{810}{y_a}.$$

TABLE LV

Step 7. Assume values of surges, y_a	8.1	10.0	15.0	20.0
Step 8. Compute K'_a from Step 6	100.0	81.0	54.0	40.5
Step 9. Compute $p = \frac{6.6 - 4.95}{6.6} = 0.25$				
Step 10. Find N'_a from Curves No. 1 of Fig. 347	78.0	56.0	34.5	25.0
Step 11. Compute F from Step 4	685.0	353.0	134.0	70.4
Step 12. Assume $d_a = 13.5$ ft				
Compute departure $= 0.422 \times 4.95^2 + 13.5 =$	23.8	23.8	23.8	23.8
Step 13. Lowest surge, $Y_a =$ Step 12 + Step 7 =	31.9	33.8	38.8	43.8
Step 14. Assume $R = A = 44.18$ sq. ft.				
Compute $D = \sqrt{\frac{F + 44.18}{0.785}}$	30.5	22.5	15.1	12.1
Step 15. Plot Steps 13 and 14 in Fig. 348.				

RETARDATION

Step 1. Let $Q = 304$ sec.-ft.

$v_2 = 6.87$ ft. per sec.

Step 2. Let $h_f = 14.0$ ft.

Step 3. Compute $c = \frac{14.0}{6.87^2} = 0.296.$

Step 4. Let, as before,

$A = 44.18$ sq. ft.

$L = 12{,}800$ ft.

Compute

$$N'_r = \left(100 \times 0.296 \times 6.87\sqrt{\frac{2 \times 32.2}{44.18 \times 12{,}800}}\right)\sqrt{F},$$

or $N'_r = 2.17\sqrt{F}.$

Sept 5. For complete shut-down, compute

$$y_r = \frac{100 \times 0.296 \times 6.87^2}{K'_r}$$

or $$y_r = \frac{1400}{K'_r}.$$

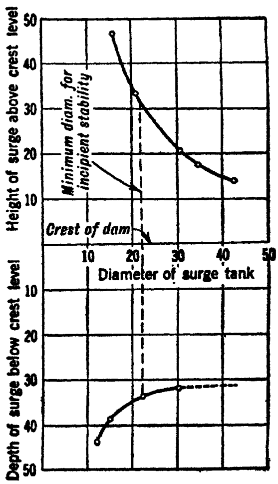


Fig. 348.—Diagram for Explanatory Example Differential Surge Tank.

TABLE LVI

Step 6. Assume values of F	160	300	700	900	1360
Step 7. Compute N'_r from Step 4	27.5	37.5	57.3	65.1	80.0
Step 8. Find K'_r from Curve No. 4 of Fig. 347	25.5	34.0	48.5	55.0	63.5
Step 9. Compute y_r from Step 5	54.8	41.2	28.9	25.5	22.0
Step 10. Assume $d_r = 6.0$ ft.					
Compute departure $= 6.0 - 0.296 \times 6.87^2$	−8.0	−8.0	−8.0	−8.0	−8.0
Step 11. Highest surge, $Y_r =$ Step 10 + Step 9	46.8	33.2	20.9	17.5	14.0
Step 12. Assume $R = A = 44.18$ sq. ft.					
Compute $D = \sqrt{\frac{F + 44.18}{0.785}}$	16.1	21.0	30.8	34.7	42.9
Step 13. Plot Steps 11 and 12 in Fig. 348.					

TEST FOR INCIPIENT STABILITY

The minimum allowed diameter for incipient stability is computed from Eq. (154). Let $R = A = 44.18$ sq. ft.;

$$L = 12{,}800 \text{ ft.};$$

$$H = 150 \text{ ft.};$$

$$c = 0.296 \text{ for retardation as before.}$$

Then

$$D_M = \sqrt{\frac{44.18 + \dfrac{44.18 \times 12{,}800}{2 \times 32.2 \times 0.296 \times 150}}{0.785}} = 17.6 \text{ ft.}$$

And, as this is to be a differential tank, use 25 per cent greater diameter, or $1.25 \times 17.6 = 22.0$ ft. This is indicated in Fig. 348.

TEST FOR CRITICAL VELOCITY

The critical velocity is found from Eq. (156), or

$$v_c = \frac{0.1}{0.296}\sqrt{\frac{44.18 \times 12{,}800}{0.785D^2 - 44.18}},$$

$$v_c = \sqrt{\frac{82{,}300}{D^2 - 56.3}}.$$

Let	$D =$ 15.00	20.00	25.00	35.00	42.40	45.00
v_c, from previous equation,	= 22.10	15.46	12.03	8.40	6.87	6.47

From these calculations it is seen that, as the v_2 used for retardation was 6.87, no correction need be made to the curve of Fig. 348 for adopted diameters less than 42.4 ft.; but, if it is desired to adopt a diameter greater than 42.4 ft., the critical velocity must be used to determine the upward surge. For instance, if it is desired to adopt a diameter of 45 ft., the surge must be computed from the critical velocity of 6.47 instead of the velocity 6.87 previously used; the velocity of 6.47, being critical, will give the greater surge.

A diameter of 30 ft. will be adopted for the tank, and the curves of Fig. 348 show that the top of the tank must be at least 21.5 ft. above crest level and the bottom 32.0 ft. below crest level, making the total height of the tank 53.5 ft.

288. Surges in the Conduit.—The excess pressure above crest level in the conduit, due to surges in the tank, will be equal to Y_r between the surge tank and a critical point on the conduit line, and thence will decrease linearly to zero at the forebay. This rule applies to a conduit of constant characteristics throughout, but is near enough for practical purposes if the area of the conduit increases towards the forebay.

To determine the location of the critical point on the conduit line,

Let l = the distance in feet from the forebay to the critical point;

a = the velocity of wave propagation in the conduit in feet per second; and

t = time in seconds required for the excess pressure Y_r to accumulate at the surge tank.

Then,

$$l = \frac{ta}{2}. \qquad (157)$$

The determination of the value of velocity of wave propagation, a, is given in Eq. (139) of Sec. 269.

The time, t, may be assumed as follows:

For a simple tank, t is so large that the distance l will be greater than the length of the conduit; hence, for such results, the excess pressure may be assumed to diminish linearly from the surge tank to the forebay; i.e., the critical point is at the surge tank.

For a restricted-orifice tank, t may be taken equal to the time required to close the turbine gates. The critical point will be closer to the forebay for this type of tank than for any other.

For a differential tank, t may be obtained from the following equation:

$$t = \frac{Y_r R}{v_2 A} + \frac{t'}{2}, \qquad (158)$$

where t' is the time required to close the turbine gates and the other letters are as indicated in Sec. 286, v_2 being used for retardation. It should be remembered that R, the area of the internal riser, may be smaller than the area, A, of the conduit in special cases. However, R may be assumed to be about 90 per cent of A for preliminary calculations.

The excess pressure in the conduit, due to surges in the tank, may occur simultaneously with the water hammer caused by the sudden acceleration of water in the riser, as explained in Sec. 279; but this does not always happen. However, it is best to add these two effects in determining the maximum excess pressure for which the conduit is to be designed.

The foregoing method applies also to the determinations of subnormal pressures in the tank for load increases. Care should be taken that the pressure in the conduit never becomes negative as, under such conditions, the conduit is likely to collapse.

289. Heating Surge Tanks.—In cold climates it is necessary to apply heat to the water to prevent freezing. The diagram of Fig. 349[7] shows the heat loss from the surface of exposed steel tanks in B.t.u. per hour per square foot of surface area per degree (F.) temperature difference between tank water and the atmosphere. The temperature of the tank water is the average inside the tank and is usually assumed to be 32°. The surface area is the entire area of water in contact with exposed steel, plus the area of the top surface. The diagram is made for a wind velocity of 12 miles per hour. Add 0.1 B.t.u. for each mile per hour above 12. In determining the temperature

[7] Drawn by L. Goff of the Factory Mutual Laboratories, Boston.

difference and the wind velocity, it should be remembered that minimum temperatures for a given locality seldom last for many hours, nor do high wind velocities usually accompany very low temperatures.

In very cold climates, it is the general practice to lag surge tanks to assist in the prevention of freezing. The accepted best practice in this respect is to attach clip angles to the wall of the tank and bolt to these angles nailing strips, so that the lagging proper will be located 6 in. away from the wall of the tank. Such nailing strips are generally cut to conform to the curvature of the tank and are placed in a horizontal position. A space of 1.5 to 2.0 in. is provided between the strips and the tank to allow circulation of air. On very large tanks where it is a simple matter to bend the lagging, the 6-in. nailing strips can be placed vertically. This latter arrangement is a somewhat simpler means of providing for the transmission of warm air around the tank. The lagging itself invariably consists of two layers of matched 1-in. lumber with building paper between.

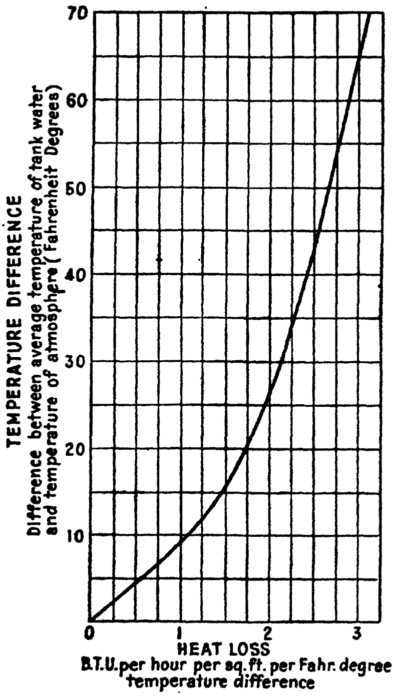


FIG. 349.—Loss of Heat from Exposed Steel Tanks for Wind Velocity of 12 Miles per Hour.

It is necessary to provide for the transmission of air into and out of the tank, on account of the changing volume of water. On small tanks, a narrow slot under the eaves of the roof is usually provided, while on larger tanks it is customary to install hinged doors, hung at the top, which are free to move in or out for the transmission of air, but which normally hang closed to confine the heat within the housing.

The usual method of heating the tank in cold weather is to use warm-air circulation between the tank and the lagging. The Associated Factory Mutual Fire Insurance Companies Inspection Department recommends, for 2.75-in. wooden tanks and for lagged tanks, one-third of the heat units shown in Fig. 349 for exposed steel tanks. However, additional heat units should be allowed for the loss of heat out of the top of the tank. In the case of flap doors and a fairly steady load condition, this loss of heat is not great; but on smaller tanks where a narrow slot is left at the top, the loss of heat may be considerable.

Ordinarily, the warm-air heating system depends on natural circulation

only, but a more positive means can be installed by using forced ventilation. Other methods of heating are in use, such as discharging warm water into the tank near the bottom. This can be further augmented by blowing air into the bottom of the tank in order to keep the contents of the tank in circulation. There does not seem to be much difference in the efficiency of the various methods of heating, provided the lagging is installed according to the standard practice. It is necessary to estimate the dissipation of heat through the lagging, and base the capacity of the heating plant on this figure. Care should be taken, in the case of the natural warm-air circulating system, to see that the air is fairly evenly distributed around the circumference of the tank and for the entire height.

In some cases the matter of fire protection is important. In the case of an elevated tank, the lagging around the riser near the base of the tank could be made of tile and the wood lagging started well above the ground level. In this way there is little danger of a fire at the ground level spreading to the wood lagging.

Heat is frequently provided at the base of the riser pipe; but, where the riser is of ample size and a slight reduction in area due to an ice coating would not be objectionable, the riser is encased but the heat is applied near its top.

Much information regarding heating appliances for tanks is given in "Specifications for Gravity Water Tanks and Steel Towers," Vol. II, published by the Inspection Department, Associated Factory Mutual Fire Insurance Companies, Boston, 1925.

For the Browns Falls surge tank, Fig. 346, the riser pipe, which is 11 ft. in diameter and about 200 ft. high, is enclosed in an octagonal framework, sheathed on the outside with two thicknesses of 1-in. cypress, planed and matched, between which is placed a layer of builder's felt. This framework is made large enough so that a series of stairs are enclosed, leading from a door at the bottom to a floor underneath the tank No heat is applied to the air space around the riser.

About 3 ft. below the bottom of the tank, there is a floor extending out beyond the lines of the tower, and from this floor a double thickness of the same cypress sheathing extends up and around the shell of the tank. This portion of the sheathing is circular in plan and is separated from the steel sides of the tank by an air space of about 8 in. This sheathing is nailed to horizontal cleats bolted to angle irons which were riveted to the shell of the tank when the tank was erected. This sheathing extends up to the eaves of the conical steel roof which is over the tank.

On the platform under the tank and around the top of the riser pipe, there has been placed a rack to which twelve heating units have been attached. Each unit consists of a G. E. Co. " Industrial Air Heater " for 750° F. operation, Catalog No. 190837G1, Type AH, Form G, 3.8 kw. The units are spaced equally around the riser and enclosed within a sheet-iron partition extending from the floor to the bottom of the surge tank, making an annular space about 30 in. wide. No trouble with freezing has been experienced, although the temperature frequently drops to 30° or 40° F. below zero.

CHAPTER XXVI

POWER-HOUSE SUBSTRUCTURE

By William P. Creager

290. Purpose.—The purpose of the power house is to support and house the hydraulic and electric equipment. In the case of very low-head plants, the power house also contains gates and racks as in Fig. 100. A discussion of gates and racks is contained in Chapter XVI.

The power house may readily be divided into two sections:

(*a*) The substructure, to support the apparatus and to provide the necessary waterways.

(*b*) The superstructure, or building, to house and protect the apparatus from the elements.

The superstructure frequently is several stories high and contains light electrical apparatus on the upper floors, as in Fig. 361. The superstructure also contains the supports for the traveling crane, which is used to handle the equipment.

Details of the power-house superstructure are described in Chapter XXVII.

291. General Arrangement.—The units are almost invariably placed in a row so as to facilitate handling by the traveling crane. The substructure is divided into a series of bays, there being a bay for each main unit, a bay for the excited units (unless the exciters are on the same shaft as the main units), and an entrance bay.

The entrance bay is at one end of the power house and provides space in which the apparatus may be unloaded and in which it may be placed when being repaired. This bay is sometimes termed the "working bay." Its size varies with the dimensions of the apparatus and is usually fixed by the size of the generator. It should be large enough to permit the entrance of a truck or freight car and to provide ample space for unloading. During the erection of the units, this space is usually filled to its utmost capacity with miscellaneous apparatus which is being assembled prior to placing in final position. If the entrance track or highway is at a higher level than the main floor, the entrance bay may be elevated, as the apparatus can readily be lowered to the main floor by the crane. Whenever possible, however, the entrance bay should be at the main-floor level.

It is necessary to provide means for removing the cores from the transformers, for repairs. The lift of the traveling crane, for this purpose, is usually greater than that required for any other purpose; and, to reduce the necessary

height of the crane and hence the superstructure, a pit is frequently provided in the entrance bay into which the transformer can be lowered before the core is lifted out. This pit is provided with a heavy cover, operated by the crane and of the same strength as the rest of the landing-bay floor. When a basement is provided in the substructure, as described later, the transformer is simply lowered through a trap-door into the basement.

Clearances in the power house between the different pieces of apparatus should be ample. There should be an unobstructed aisle 6 or 8 ft. in width from one end of the power house to the other. It is usual to draw two imaginary lines for this aisle and to locate the edge of generators, governors, stairways, etc., on these lines. This improves the appearance of the aisle as the sides are then well defined.

Many companies have adopted, as standard, a minimum of 4 ft. clearance between all pieces of apparatus outside of the main aisle. Exceptions, of course, are made to this rule in cases where such space is very infrequently used by operators.

For large installations, requiring both an operating and a maintenance force, the switchboard may be located at any convenient place, and is sometimes at quite a distance from the generating apparatus. For smaller stations, where only one or two men are required for operation and maintenance, the switchboard is generally located as near as possible to the main level and at the center of maintenance activities. Governors, pumps, and other small apparatus should be so arranged that a tour of inspection, from the switchboard to all points requiring attention, and return, will be as short as possible. This feature requires careful study in complicated stations.

If separate exciters are used, they should be located as close as possible to the switchboard, so as to minimize the amount of copper in the exciter leads. This applies also to motor-generator sets when used for excitation.

Space in the building should also be provided for the telephone booths and toilets, and for an office and a storeroom, if required. An office is required only in the very large plants where a large number of operators, with a general superintendent, are employed, and for plants that are used as the control point of a series of remotely controlled plants. The telephone booth should be close to the switchboard.

Special storerooms are required only for very large plants and those that are to be used as headquarters for transmission-line maintenance. Usually there is ample space in the power house for the storage of supplies, particularly if the general layout which is adopted includes provision for a basement.

292. Types of Substructures.—It is impossible to describe in detail the many different types of substructures that have been built. An attempt will be made, however, to cover the general theory of the usual modern types, cf which there are, of course, many variations to suit local conditions and requirements and the ideas of the designer.

The modern power-house substructure is built exclusively of concrete, with a small amount of steel work to support the lighter floors. The general types of substructures may be classified according to the type of turbines adopted. In general, they include the following:

(a) For open flumes;
(b) For vertical concrete spiral casings;
(c) For vertical metal spiral casings;
(d) For horizontal metal spiral casings;
(e) For impulse turbines.

The various types of turbine casings are described in Chapter XXVIII, and the conditions governing the choice of type are stated in the same chapter. The dimensions of the open flumes and spiral casings are always fixed by the turbine manufacturers, although slight changes may be made by them to conform to the necessities and desires of the engineer. The arrangement having been decided upon, drawings showing dimensions are supplied by the turbine manufacturer.

293. Open-flume Settings.—The Sewells Island installation, shown in Fig. 350, is a typical open-flume setting. The headgates may be inside or outside of the substructure. At Sewells Island they are located inside the substructure and handled by the power-house crane. Care should be taken that high-water surface, particularly that caused by surges in high-velocity canals,[1] is lower than that elevation which will cause damage to the installation above the turbine. The generator leads and the passageway to the steady bearings near the main-floor level are usually the governing features. The downstream wall of the substructure is sometimes carried only as high as ordinary high-water surface, thus providing a spillway to prevent excessive surges from high-velocity canals. The support for the generator, spanning the open flume, should be designed for the weight of the generator, the shaft, the runner, and the hydraulic thrust. Ample allowance should be made for impact and vibration, as surges in the draft tube will cause rapid fluctuations in the hydraulic thrust. The maximum allowed deflection of the support is also fixed by the clearances in the turbine runner. The support should have the approval of the manufacturers of the hydraulic turbine and of the generator.

294. Vertical Concrete Spiral-case Setting.—The installation shown in Fig. 351 is a typical vertical concrete spiral-case setting. The concrete over the draft tube must be reinforced to sustain the loads above it. Frequently the draft-tube design will allow the installation of a pier in the center of the draft tube to reduce the span of this reinforced concrete. The weight of the generator, the shaft, and the runner, and the hydraulic thrust, is carried through the speed ring of the turbine to the concrete around the draft tube.

The concrete spiral casing is in reality a pipe; and, under the higher heads, considerable steel reinforcement must be provided to take upward pressure on the top of the casing. This upward pressure is tied to the base of the spiral casing where it is balanced by the downward pressure. It is permissible to figure on the weight of the concrete in balancing part of the upward pressure, but the generator may not be in place when the casing is under full head. Water hammer [2] should be included in the upward pressure.

The upward pressure should be carried to the base, both through the walls of the casing and through the speed ring of the turbine. This should be

[1] See Sec. 78.
[2] See Chapter XXIV.

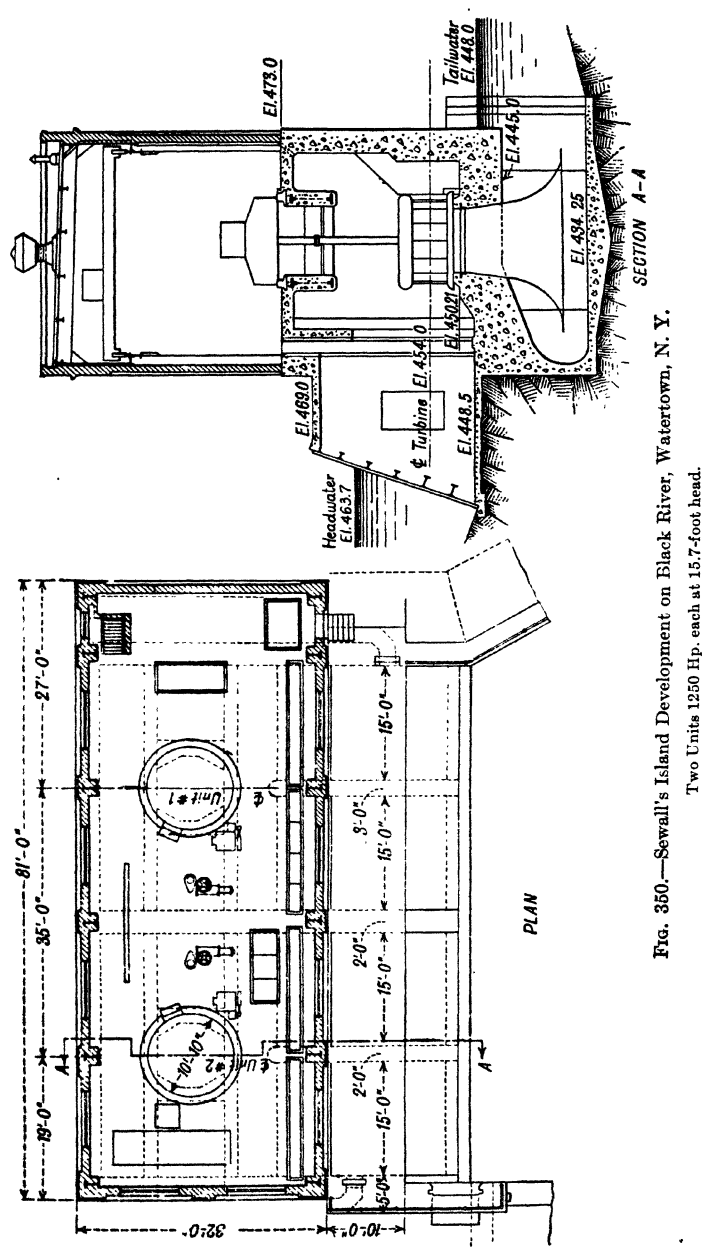


FIG. 350.—Sewall's Island Development on Black River, Watertown, N. Y.

Two Units 1250 Hp. each at 15.7-foot head.

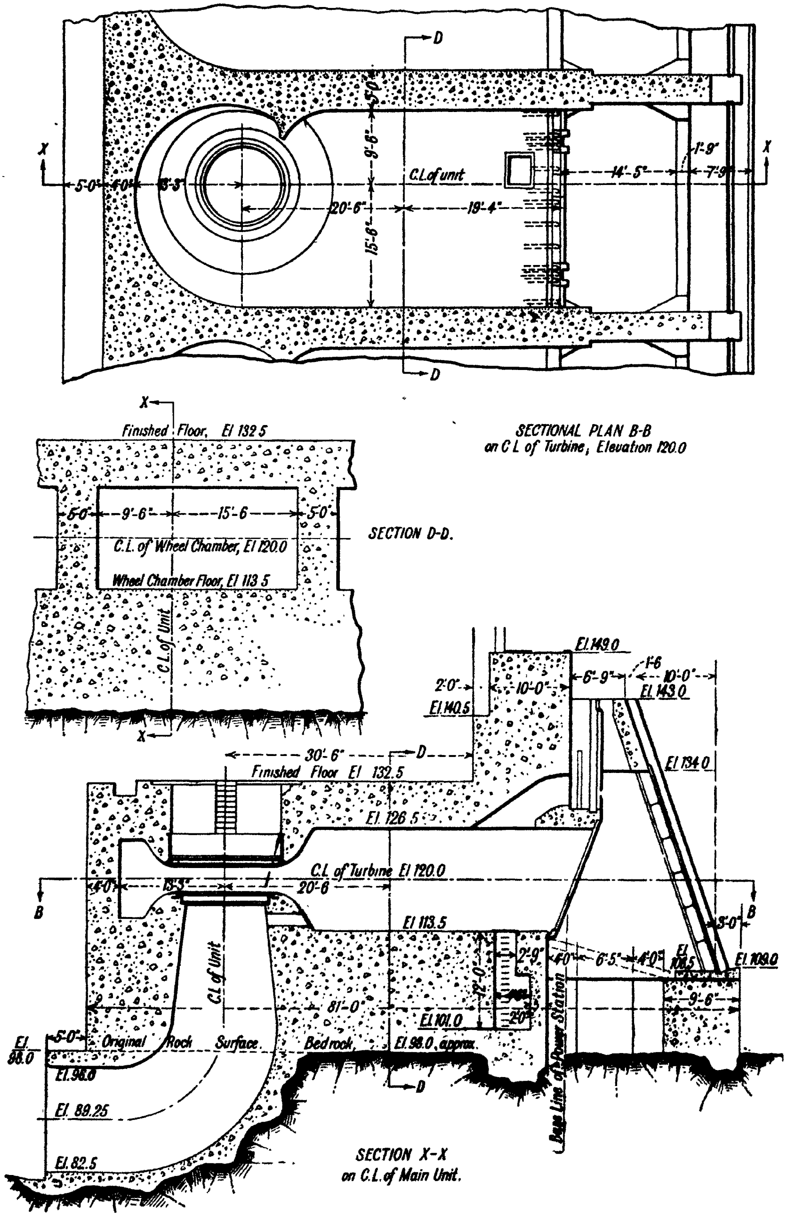
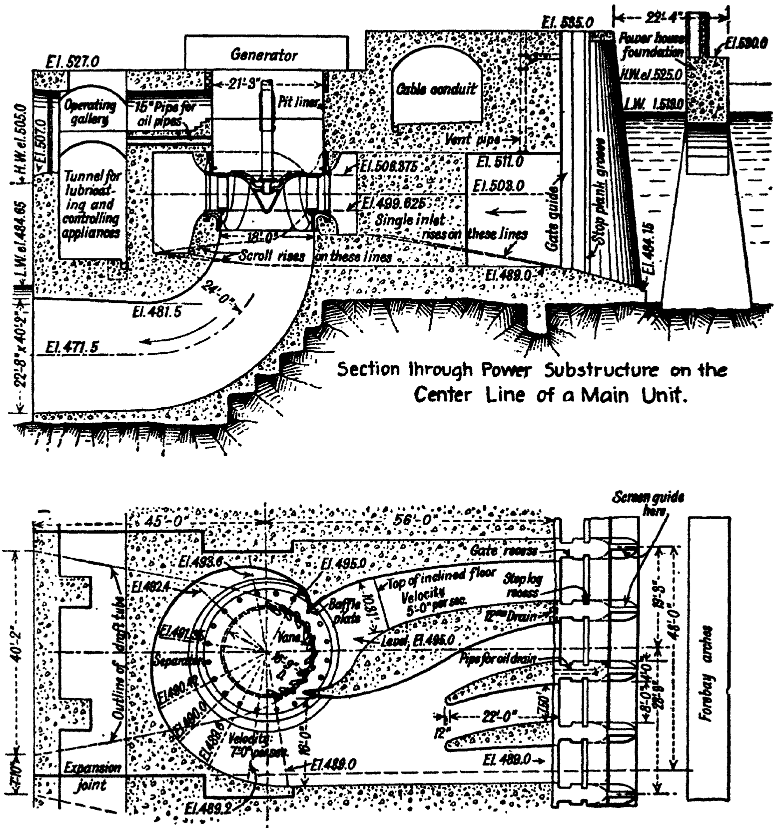


FIG. 351.—Power House Substructure of Parr Shoals Power Co.

insisted upon, because the slab, against which the upward pressure will occur, cannot span between the casing walls without upward deflection, and such deflection will be resisted by the concrete above and below the speed ring of the turbine and may cause cracking unless this concrete is properly reinforced. Therefore, a positive tie should be provided through the speed ring of the tur-



Plan of Intakes and Draft Tube for a Main Unit.

FIG. 352.—Power House Substructure of Keokuk Development, Mississippi River Power Co.

Engineering Record, Vol. 66, p. 538.

bine. Very wide spiral casings may have partition walls, as in Fig. 352, to reduce the span of the top of the casing and the width of the intake gates.

Entrance to the pit over the turbines is by a stairway, usually located at the small part of the spiral casing. In most cases the generator must be supported on a hub of concrete, as in Fig. 100, in order to provide head room under the stator frame.

295. Vertical Metal Spiral-case Settings.—Various methods are used to support vertical turbines having metal spiral casings. Such casings, being

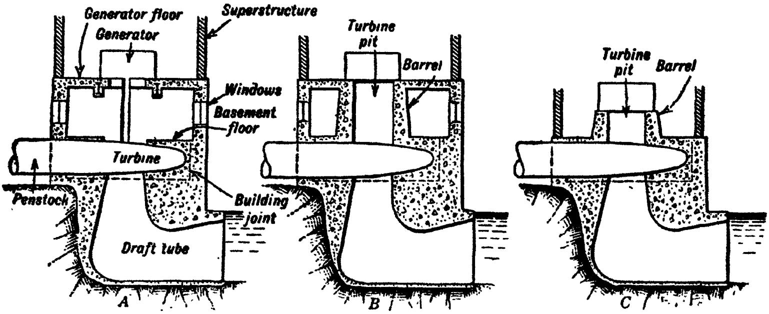


FIG. 353.—Typical Arrangements of Substructure for Vertical Metal Spiral Case Units.

self-contained, make the design of the substructure less intricate than in the case of concrete spirals. Aside from the necessary provision of a simple support for the unit, there must be sufficient concrete around the spiral to act as an anchor for the end of the pipe line.

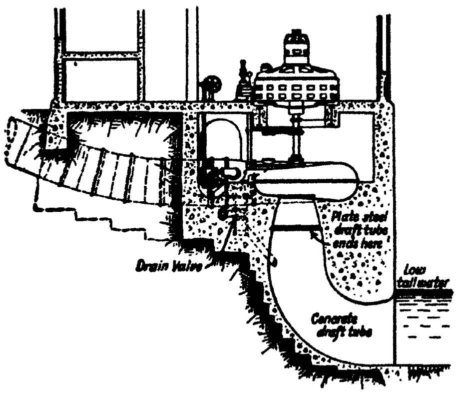


Fig. 354.—A Typical Type *A* Setting. S. Morgan Smith Co.

An arrangement similar to this was used at the Niagara Mill of the Kilberly-Clark Co., Wisconsin.

Several examples, showing the types usually adopted for substructures of this class, are shown in the diagrammatic sketches of Fig. 353.

In Type *A* the generator, shaft, runner, and hydraulic thrust are supported entirely by the generator floor. The support for the generator should receive careful attention, as previously explained for open-flume settings. This type is adapted to small units only, as it is difficult to obtain rigid supports for large generators without considerable expense. Figure 354 is a typical example.

In Type *B* the entire weight is carried through a concrete barrel and the turbine speed ring, to the foundations. Figure 355 is a typical example.

Type *C* is similar to Type *B* except that the basement is omitted and the generator sets on a concrete barrel at some distance above the floor, in order to provide head room for entrance to the turbine pit. Figure 356 is a typical example.

The advantages of a basement, as in Types *A* and *B* are as follows:

(*a*) The generator is at the floor level.
(*b*) Outside air is available, through the basement windows and the openings in the concrete barrel, for ventilation of the generator. This

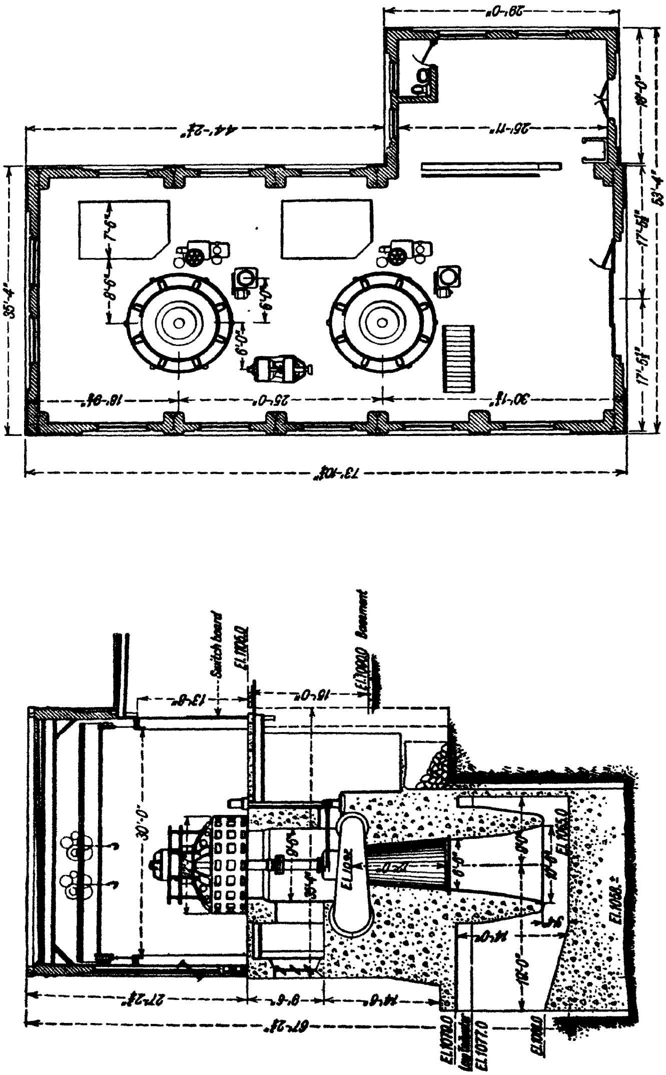


FIG. 355.—Browns Falls Power House of the Northern New York Utilities, Inc.

effect could, of course, be obtained in Type *C* by an air duct from the turbine pit, over the floor and through the superstructure wall

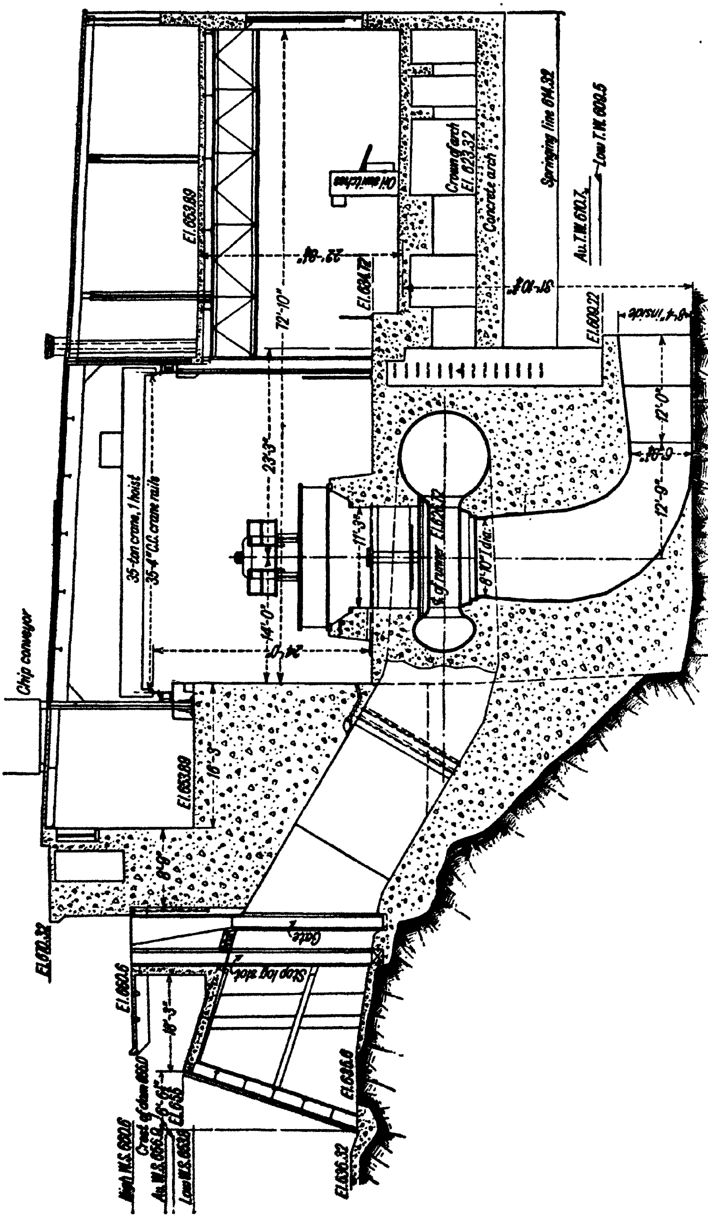


FIG. 356.—Deferiet Power House on Black River at Deferiet, N. Y. The Power Corporation of New York.

to the outside air; this has, in fact, been done, but it places an obstruction on the floor, which is objectionable. However, in certain cases, sufficient ventilation may be obtained for Type *C* by circulation of the inside air through the barrel and generator.

(*c*) A basement provides room for storage purposes and for the installation of electrical conduits and oil and water piping on the ceiling.

The disadvantages of a basement are as follows:

(*a*) Increased cost of the substructure, due to the additional floor. However, the superstructure is higher for Type *C* than for the basement type, by the height of the barrel in Type *C*.

(*b*) A single stairway is usually provided for access to the basement, unless the number of units is great; whereas for Type *C*, an easier entrance is provided through the barrel at each turbine pit.

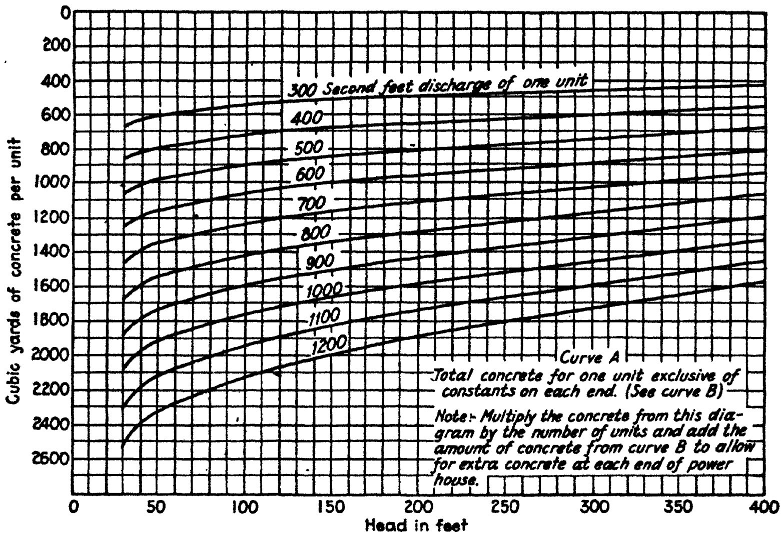
The disadvantages of having the generator at a greater distance above the floor, as in Type *C*, may be partly overcome by providing a bridge between the tops of each two adjoining generators so that an operator, on his tour of inspection, can pass directly from one generator to another. Sometimes, for Type *C* and also for the other types, if the generator is very high, the switchboard is placed on a gallery at the level of the tops of the generators, and bridges are provided between the generators and the switchboard gallery. This places the main operating level at the top of the generator, and the operator descends to the main floor only to attend to the governors and other accessories.

Ample head room should be adopted for the basement because, as the design progresses, it will be found desirable to attach to the ceiling many items of electrical equipment which may not have been thought of when work on the design was begun.

It is very desirable to arrange for the completion of the superstructure and the installation of the traveling crane before the erection of the spiral casing has been started. For this reason a building joint is usually provided as shown on the sketches of Fig. 353 in order that the concrete around the spiral casing can be placed at a later time. This necessitates a greater distance between the spiral casing and the down-stream face of the substructure than would otherwise be necessary, as room must be provided between the building joint and the casing to permit riveting. It is not necessary, in any of the foregoing types, to encase the spiral casing entirely in concrete. In many instances the floor is at or near the level of the center line of the casing. However, this feature breaks up the continuity of the floor and should not be adopted under any consideration for Type *C*. It is quite usual to have the large part of the spiral project slightly above the basement floor, as the other side of the basement then provides a continuous and level passage.

The curves in Fig. 357 are for use in making rough preliminary estimates of the quantities of concrete required in power-house substructures, of the type indicated in Fig. 356, having steel spiral-cased turbines. These curves were prepared for this book by Mr. F. H. Burnett, from data supplied by the various manufacturers of turbines. However, they may not be accurate for

any given case, as two power houses designed for like conditions by different engineers would differ in the quantities involved. The curves have been



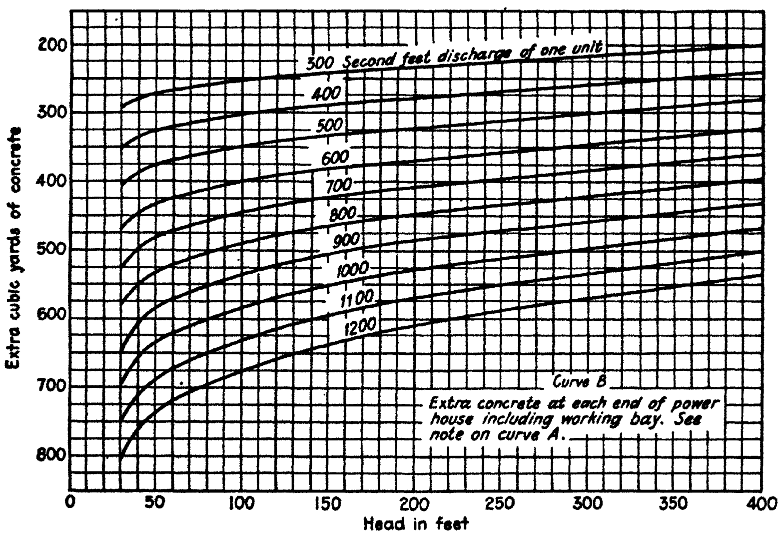


Fig. 357.—Approximate Power-house Substructure Concrete for Type *C* Steel Spiral Case Vertical Units.

compared with a number of existing designs and found to agree closely enough for preliminary estimates.

Curves A give the quantities of concrete for one unit. The quantities from Curves A should be multiplied by the number of units, and a constant from Curves B added to allow for extra thickness of end walls, for working bay, and for other incidentals. Curves B do not include the concrete necessary for penstock valves, if used, or other special features.

Example:

Discharge per unit,	1000 sec.-ft.
Head at full load,	75 ft.
Number of units,	3
Concrete from Curves A,	$1812 \times 3 = 5436$ cu. yd.
Concrete from Curves B,	609 cu. yd.
Total concrete in substructure,	6045 cu. yd.

296. Horizontal Metal Spiral Casing.—Figures 358 and 359 show typical examples of substructures for horizontal units with metal spiral casings. Figure 358 is for individual penstocks, and Fig. 359 for a single penstock. By lowering the penstock so as to provide a vertical riser pipe between the penstocks and the spiral casing, the units may be revolved about the vertical riser and made to occupy any desired position. The arrangement in which the axis of the unit is at right angles to the center line of the penstock, as shown, is the usual one, as it provides for a higher penstock and usually less cost. The units can be revolved also by providing a horizontal bend in the penstock of Fig. 358, just before it joins the spiral casing. It will be noted that the arrangement shown in Fig. 358 provides for a long, narrow power house, while that in Fig. 359 requires a short, broad structure. This feature deserves consideration where the shape of the available space influences the shape of the power house.

297. Substructure for Impulse Turbines.—Figure 360 shows a typical substructure for an impulse turbine. The details are somewhat less complicated than those for other types of turbines, and are fixed mainly by the requirements of turbine and generator builders. No special features of design can be discussed in this instance, as the types of installations vary greatly according to requirements.

298. Draft Tubes.—Draft-tubes types, described in Chapter XXVIII, are usually fixed by the recommendations of the turbine manufacturer, who furnishes detailed dimensions. The manufacturer is usually willing to conform, within certain limits, to the desires of the engineer, in order to meet special requirements of the substructure design. However, it is necessary to follow exactly the final dimensions furnished by the manufacturer in order that his guarantees may not be invalidated.

There are at least as many general types of draft tubes as there are manufacturers of turbines, and most manufacturers have several types which they recommend according to the type of turbine.

There seems to be no standardization in draft-tube design, as the types recommended by the various manufacturers differ widely. This is probably

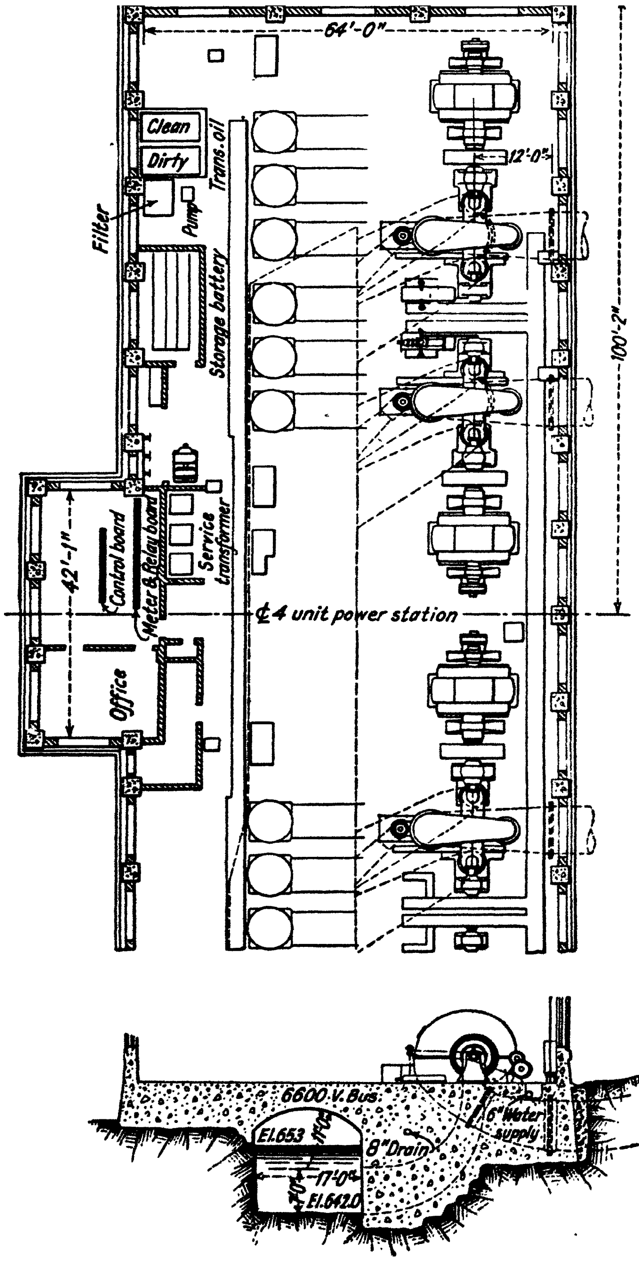


FIG. 358.—Horizontal Units of the Salmon River Development of the Salmon River Power Co.
Eng. Record Vol. 69, p. 671.

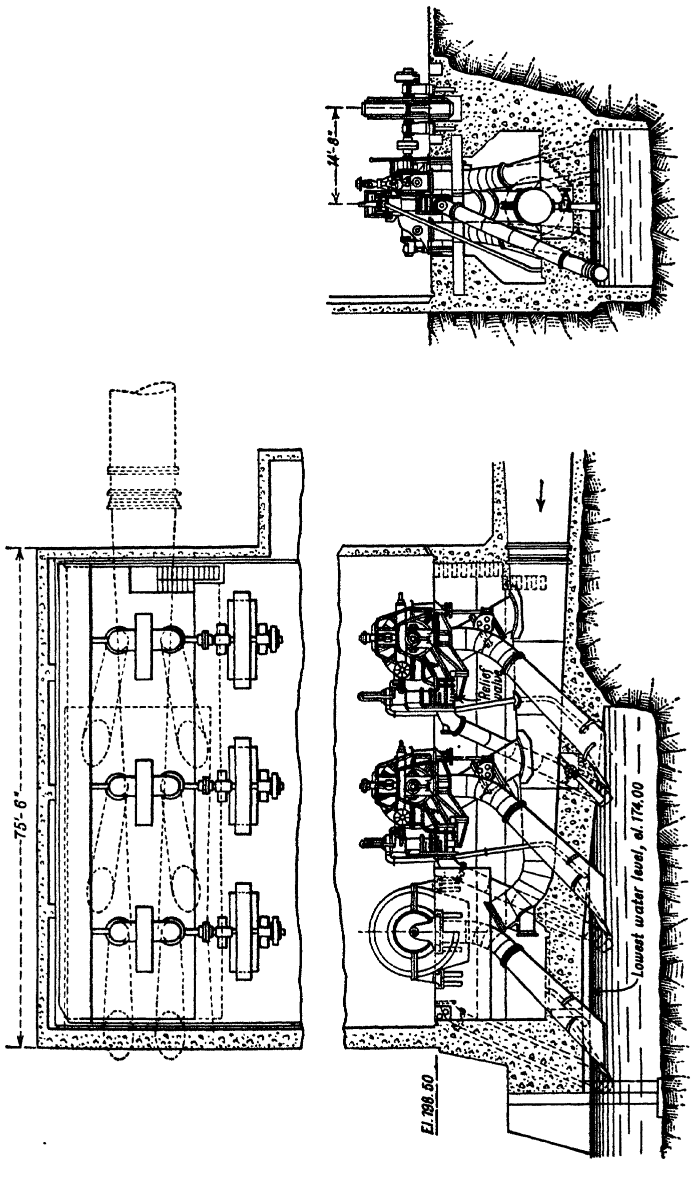


FIG. 359.—Horizontal Units of the Grand Falls Development of the Anglo-Newfoundland Development Co., Ltd.

Eng. Record, Vol. 64, p. 93.

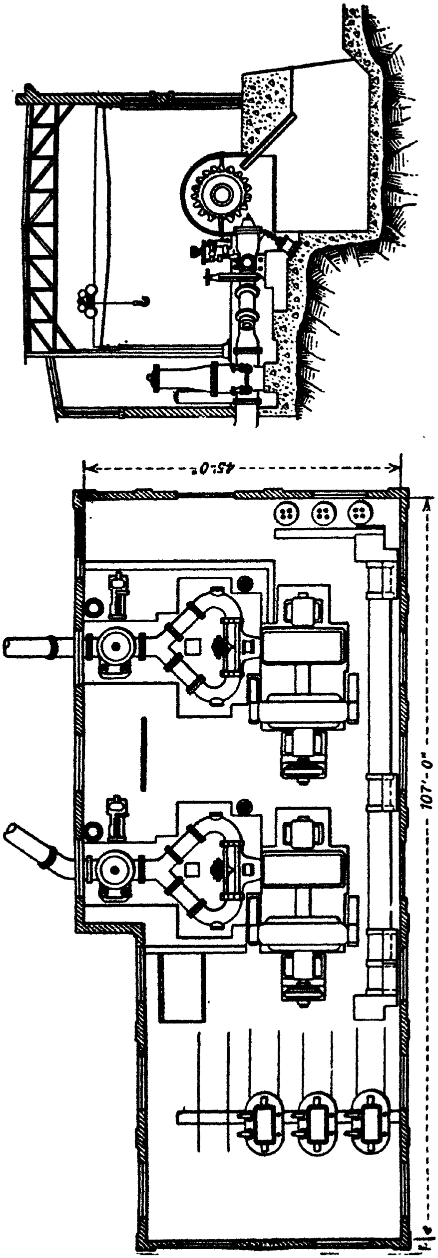


FIG. 360.—Plan and Cross Section of Power House of Boulder Hydroelectric Development. Head, 1830 ft. Capacity Each Unit, 10,500 H P.—Eng. Record Vol. 62, p. 118.

because sufficient experiments have not been made under varying conditions to indicate closely which type is best. Some types are considerably more expensive to construct, and require a larger substructure than others. Consequently, the engineer, unless he has a decided preference, must give cognizance to this increased cost in the selection of the turbine.

299. Conduits. — Electrical conductors may be carried in a specially provided gallery in the substructure, on the walls or ceiling of the basement, or in conduits in the floor. It is seldom that the number of conductors is sufficient to require a special gallery to contain them, unless such gallery can be provided at small cost because of the arrangement of the substructure. If the cables are carried on the ceiling of the basement, ample head room should be provided to allow for the necessary supports.

Iron or fiber conduits may be buried in the concrete of the substructure, and the conductors drawn through them. In such cases the conductors are protected by a lead cover or other weatherproof material. The conduits should always be drained if the lead cover is not used. Large conductors are usu-

ally carried in fiber conduits, and it is best to provide pits in the floor at each sharp bend in order to facilitate pulling. Standard pull-boxes are used for the smaller iron conduits.

Wiring for the control, exciter, and other small pieces of apparatus is usually run in iron conduits in the concrete, just below the finished generator floor surface. These wires are frequently very numerous and a number of crossings are required. The floor is so designed as to have sufficient strength without the top layer of concrete, which is omitted temporarily. After the ducts have been laid and the wiring installed, a final layer of concrete, of the necessary thickness, is placed. This layer is frequently poured after the plant is in operation. From 6 to 12 in. is required for such conduits, depending upon the size and number of crossings. Fiber ducts must be embedded in the concrete before the conductors are pulled, and are sometimes placed in the concrete below the layer containing the smaller wiring.

Care should be taken that the fiber ducts, which are structurally weak, do not interfere with the stability of reinforced concrete members. Iron ducts, unless too thickly placed, will not weaken the floor.

Careful plans should be made of the proposed layout of conduits, as this cannot be done advantageously in the field after the general design is completed.

300. Foundations.—The substructure may be placed on any type of foundation that is suitable for other buildings. The preparation of soft foundations to receive the structure differs in no way from standard practice except as to the necessity of preventing erosion, which may be caused by the water issuing from the draft tubes. If this seems likely to occur, a concrete apron should be provided and extended far enough down-stream from the power house to insure safety from scour. Sheet piling should be provided at the end of the apron, or other measures taken to prevent its being undermined. Riprap is sometimes used for this purpose. On account of the vibration caused by the apparatus, it is best to use about one-half the allowed bearing pressures usually adopted for static loads.

301. Ventilation of the Generators.—The subject of ventilation of the power house, to prevent overheating and to supply cool air to the generators, is very important. It is, however, a subject that requires very expert attention, as the problem differs with each installation and depends upon the climatic conditions, the type of substructure, and the details of construction of the generators. Small installations in cool climates may be sufficiently ventilated if provision is made for circulation of air through the generator, and may not require a continuous supply of outside air. Large units in warm climates frequently require somewhat elaborate provisions for large ducts, through which outside air may be admitted to the generators. The movement of the air to the generators is caused by fan vanes placed on the rotor; or, if this is not sufficient, motor-operated fans may be used in the air ducts.

Unless the designer has special knowledge of this subject, he should freely consult the experts employed by the generator manufacturer, before adopting the final layout of the substructure. Several methods of ventilation, adapted to the usual type of substructure, are described briefly in Sec. 295.

302. General Details.—Data on the generator torque should be obtained from the manufacturer. Provision should be made to prevent twisting of the generator in case it is supported on a high concrete barrel, as in Sketch *C* of Fig. 353, and to prevent overturning of a horizontal generator if not firmly anchored to solid foundations.

The substructure is usually made entirely fireproof. This provision is frequently demanded by economy in view of the lower fire insurance rates. Small doors and windows may be of wood, provided they are not adjacent to apparatus which is likely to take fire.

All small pieces of apparatus should be placed on concrete pedestals about 6 in. high, both to improve their appearance and to prevent possible damage by water on the floor.

The main operating floor of the power house has been variously treated. Various kinds of floor-hardener preparations are available and should be used to prevent accumulations of dust. The best type of hardener is that which can be applied by a brush or broom after the floor is finished. Where an artistic appearance is desired, tile and mosaic floors have been used; these add considerably to the appearance of the power house and provide a surface which is easily cleaned.

Certain rules for details of design for safeguarding the power-house operators are mandatory in some states. These rules pertain to fire doors; details of stairs, ladders, and walkways; clearances; railings, and other similar items. They can be obtained through the insurance companies and contain many valuable suggestions.

Oil tanks should be placed in the basement or, preferably, buried under ground. They should be drained to the tail race by the operation of a valve which would be accessible in case of fire.

CHAPTER XXVII

POWER-HOUSE SUPERSTRUCTURE

By Joseph H. Gandolfo

303. General Conditions.—The primary functions of the superstructure of any power house are to shelter and protect the machinery and operators and to provide facilities for handling the equipment. Thus the ultimate use of the superstructure of a hydro-electric power house does not differ from that of any other industrial building. There are, however, many factors that must be taken into consideration in the design of such a building, and these often considerably modify the merely box-like structure that otherwise might be built.

The layout of the power-plant equipment, including turbines, generators, exciters, switchboard, oil switches, and other auxiliary machinery, and sometimes high-tension transformers, determines the general over-all dimensions of the building. As a rule, the clearance for the traveling crane, necessary to handle the largest pieces of equipment and carry them over the other machinery while it is in operation, determines the height of the building. In particular cases, however, other considerations may govern the height of the building, as in Fig. 350, where the headgates, located inside the power house and lifted by the crane, are the controlling influenec.

The following is a list of the usual equipment for which space must be provided in the superstructure:

(1) Main generating machinery;
(2) Turbine governors, pumps, and tanks;
(3) Motor-generator sets;
(4) Compressed-air equipment;
(5) Water-supply pumps;
(6) Switchboard and low-tension switches and buses;
(7) High-tension transformers and switches (if not in an outdoor substation);
(8) Storage batteries;
(9) Transformer oil tank with filter and pumps;
(10) Telephone or radio equipment;
(11) Storage space for miscellaneous supplies, such as oil, waste, spare parts, line material, etc.;
(12) Lavatory;
(13) Office.

In addition to housing the equipment and auxiliaries, space must be provided on the main floor, at one end of the building, for handling and disman-

tling the machinery. This space should be located at the entrance to the power house, and is ordinarily spoken of as the "landing bay." For a further discussion of this, see Chapter XXVI.

If a basement is provided in the substructure, as described in Chapter XXVI, less space will be required on the main floor, as some of the less-used auxiliary equipment may be located in the basement.

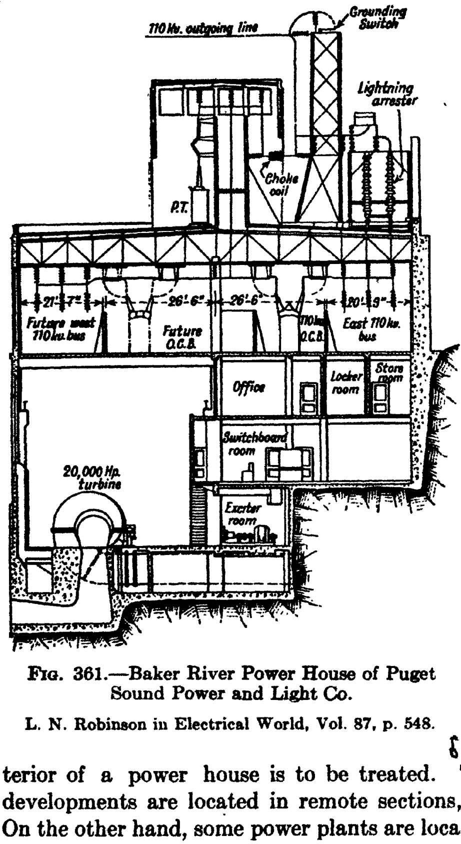


FIG. 361.—Baker River Power House of Puget Sound Power and Light Co.

L. N. Robinson in Electrical World, Vol. 87, p. 548.

It is frequently necessary to provide an ell or lean-to on the main building, as in Fig. 355, to obtain sufficient space for the switchboard and switches. In some large power houses, where the high-tension electrical equipment is located inside the building, as in Fig. 361, and space is limited, it is necessary to provide several stories in the superstructure. However, this is very infrequently the case, except for very large installations, particularly when the high-tension equipment is located in an outdoor transformer station, as is usual. Consequently, most superstructures are one-story buildings.

304. Architectural Effect.—It is sometimes very difficult to decide how the exterior of a power house is to be treated. The majority of hydro-electric developments are located in remote sections, far from main lines of travel. On the other hand, some power plants are located in or near centers of population; and some of those located away from such centers are so situated that architectural effect is an important consideration. The advent of the automobile has brought many remote plants within easy visiting distance of the public. Thus it becomes more and more important to select a pleasing architectural effect for the superstructures of power houses.

It is not necessary to design a power house without making any attempt to obtain a pleasing result. In Europe, far more consideration has been given to architectural effect in power-house design than in this country; and, on account of cheap labor and materials, highly ornamental structures of dimension stone are common abroad.

However, there is no necessity of going into such expensive designs to

obtain a sturdy and handsome building. Many excellent examples exist in this country of good-looking structures built of reinforced concrete, brick and other materials; and such exteriors can be built with very little additional outlay over what a purely utilitarian superstructure would cost. The writer has obtained very good-looking superstructures by using common brick ornamented with a small amount of pre-cast concrete.

The accompanying illustrations show photographs of several types of superstructure exteriors.

305. Special Architecture.—A hydro-electric power house is a purely utilitarian structure. It is sometimes considered necessary, not merely to erect an ordinary good-looking building, but to build a highly ornamental structure, both inside and outside. In such cases, cut stone or terra cotta trim may be

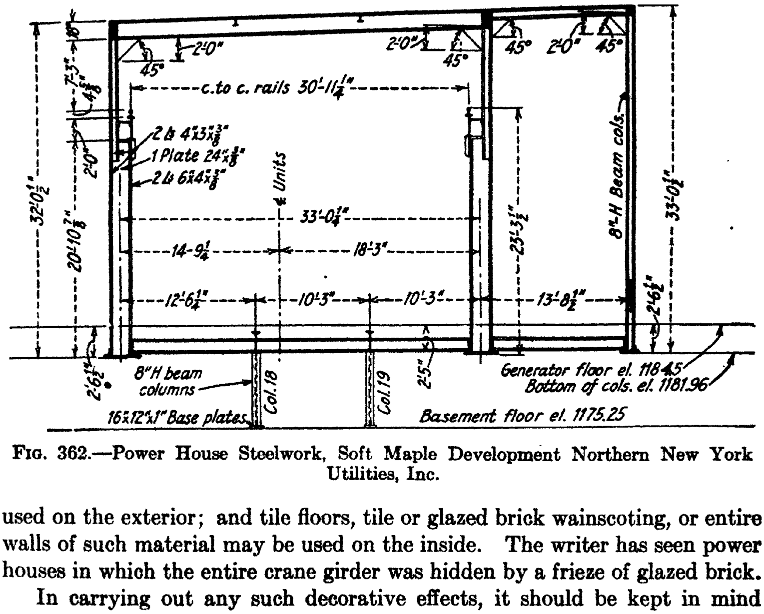


FIG. 362.—Power House Steelwork, Soft Maple Development Northern New York Utilities, Inc.

used on the exterior; and tile floors, tile or glazed brick wainscoting, or entire walls of such material may be used on the inside. The writer has seen power houses in which the entire crane girder was hidden by a frieze of glazed brick.

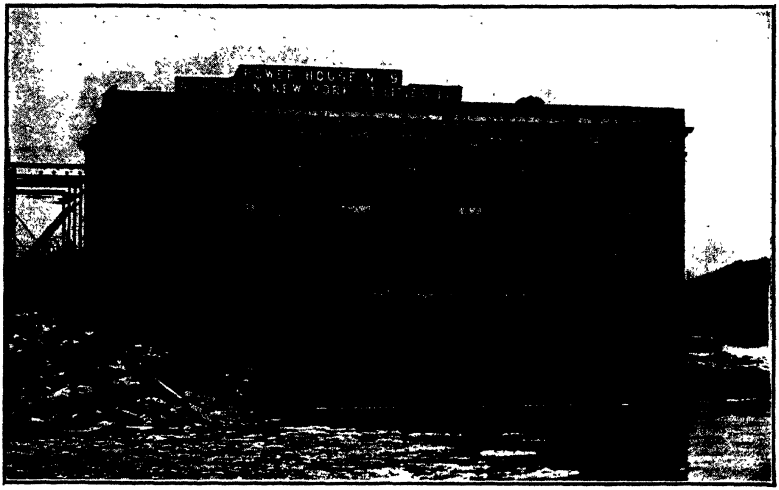
In carrying out any such decorative effects, it should be kept in mind that they should give the effect of strength, solidity, and massiveness, and that this should be the controlling *motif* throughout. The light, airy, and fantastic has no place whatever in a power-house design.

306. Framework.—Except for very small developments, the framework of the power-house superstructure should unquestionably be of structural steel. Some power houses have been built with heavy walls supporting the crane runway and the roof, but there are so few of these that they may be considered almost negligible.

Figure 362 is a typical cross-section of the steel work for a one-story power house.

307. Walls.—All power houses of any consequence are built entirely of fireproof construction, and walls of the following materials may be used:

(1) Stone work, backed with brick or other material;
(2) Brickwork;
(3) Brick veneer, backed with other material;
(4) Reinforced concrete;
(5) Concrete blocks;
(6) Hollow tile (terra cotta blocks);
(7) Thin plastered walls on steel framework.



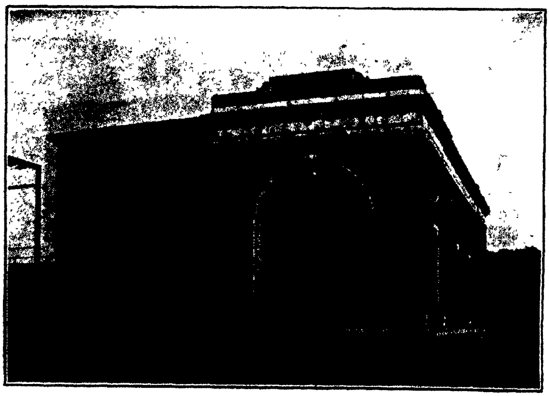


FIG. 363.—Black River Plant of the Northern New York Utilities, Inc. A Highly Ornamental although Expensive Brick and Stone Exterior.

It is often difficult to make a decision as to the best and most economical material to be used for the superstructure. In addition to the facts that can

be determined beforehand, there are many factors that at best are very uncertain. Such, for example, are the exact cost per ton-mile for hauling materials to the new site, the exact time of year at which the superstructure will be built, and the condition of the roads at times of heaviest hauling.

Stone work, either dimension stone or rubble work, can be dismissed with the remark that such material should rarely, if ever, be used in private developments. It adds greatly to the cost of the superstructure, and the only excuse for using it would be the fact that the superstructure must harmonize with its surroundings. An exception to this might occur, however, where suitable stone, already well broken up, existed at the site of the power house. Even then it would probably be more economical to crush the stone and build the structure of concrete or concrete blocks.

The other materials mentioned all have their proper places. Brick can be used to very good advantage where the haul is not too long, the power house not too large, and particularly where the building must be erected during severe winter weather. By heating the brick, and the sand and water for the mortar, and keeping the mortar in the tubs warm by means of specially designed mortar tubs with lamps or steam coils under them, brickwork can be laid up in a temperature many degrees below the freezing point and a perfectly good wall obtained. This work can be carried on at temperatures that would make it impossible to build reinforced concrete walls, to lay up concrete or terra cotta blocks, or to build plastered walls, without elaborate and costly methods of protecting and heating the structure.

Another advantage in the use of brick is that a very good-looking and smooth wall can be secured on the inside of the building. There is no need of plastering such a wall, and no great need of painting it. Painting can be done at any time in the future if the walls become dingy, or if it is desired to brighten up the interior.

Brick-veneered walls can be used, the backing being pre-cast concrete blocks, terra cotta blocks, or reinforced concrete. The only advantage of such walls over solid brick walls is that they make a good appearance on the exterior with a minimum of expense. Care must be taken to anchor the brick thoroughly to the backing so that the front will not separate from the main wall.

With this type of construction it is rarely possible to obtain a smooth and good-looking wall on the inside; for this reason, if inside appearances are to be considered, the interior surfaces of the walls must be plastered. In considering any kind of plaster work, however, one should bear in mind that it is very difficult, and often impossible, to get plasterers to go to out-of-the-way locations, without excessive cost for transportation and bonuses. This applies to all plaster or stucco walls that otherwise might be built in connection with a power house.

If the haul from the railroad is long, and the location far removed from centers of the brick-making industry, a reinforced concrete or concrete block superstructure may be advisable, provided sand and good gravel or stone are available near the site.

If the building is a large one and it is, therefore, possible to use wall forms a number of times, reinforced concrete walls may be the cheapest type of con-

struction. Figure 366 shows a good example of this type of construction. If concrete is used, provision must be made for expansion joints; otherwise, unsightly cracks are sure to develop in the walls. For a good description of the reinforced concrete superstructure of the Cedars Rapids power house, see *Eng. Record*, Vol. 70, p. 107.

If care is taken in the design of such a building, a fairly good-looking

Fig. 364.—Effley Falls Power House of the Northern New York Utilities, Inc. A Simple but Pleasing Brick Exterior.

structure can be obtained, although a concrete or concrete block structure cannot equal the appearance of a brick building.

If the haul is long and the building too small to warrant the use of reinforced concrete walls, pre-cast concrete blocks may be used. Figure 368 shows a good example of this type of construction.

If concrete blocks are decided upon, the walls must be carefully designed in detail and the number of each kind of block necessary to form window and door jambs, corners, lintels, etc., determined upon. Enough additional blocks must be made to cover breakage, and any other contingencies that may arise on the job.

The concrete blocks should be cast at least one month before they are to be used in the walls. As, for a small power house, it would rarely be economical to have more than one block machine on the job, the work of casting the blocks must be started in ample time, so that practically all blocks are completed a month before the superstructure is started. Blocks must not be made in freezing weather, unless a heated structure is provided to house the machine. Blocks must be protected from freezing and the weather, until thoroughly set up and at least partly seasoned.

A perfectly good and lasting wall can be built of concrete blocks, provided the blocks are properly made. Sufficient cement must be used to make a good and fairly rich concrete or mortar for the blocks. Commercial blocks very

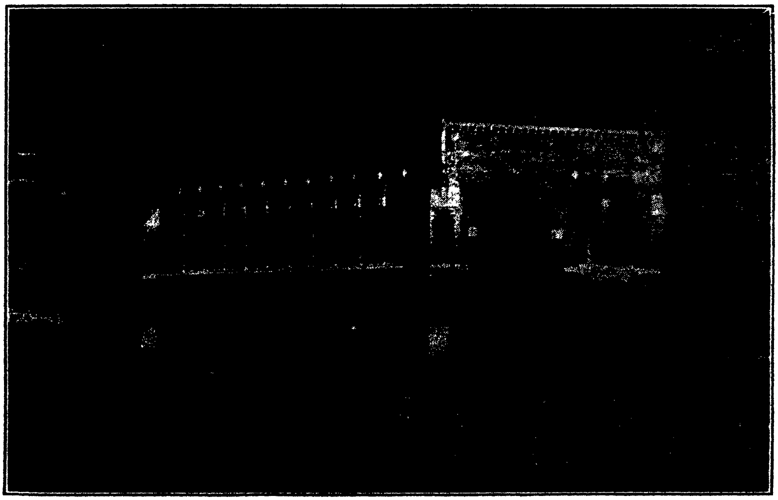


FIG. 365.—Davis Bridge Power House of the New England Power Co. A Brick Exterior of Appropriate Design.

often are not well made, the writer having seen such blocks that could be kicked apart with the foot.

Although a pleasing architectural effect can rarely be obtained with concrete blocks, the exterior face of the walls can be laid up in a neat and workmanlike manner. It is, however, practically impossible to obtain a good-looking wall on the interior of a concrete block building, if the blocks alone are used. If a neat appearance on the inside is desired, it is necessary to plaster the walls, thus adding to the cost of such walls an item of expense which must be balanced against other types of construction.

If appearances are not to be considered and the building is reasonably near a railroad, hollow tile can be used for the walls. Such tile must be heavy exterior wall tile, made especially for the purpose. Stucco finish on hollow tile, as in Fig. 369, makes a neat appearance.

Here again, the number of tile for jambs, corners, etc., must be carefully

determined beforehand and sufficient allowance made for breakage in transit and on the job.

It is claimed that it is cheaper to lay up a wall of hollow tile than one of brick. Theoretically, this is so, but it does not always work out so in practice. It is much more difficult to build up the hollow tile around windows, doors, etc., and fit them around columns and other steel work, than it is to lay up brick work. Furthermore, with the present difficulty of obtaining good and efficient brick masons, particularly in remote localities, the laying up of tile blocks is not as speedy as might otherwise be the case under different conditions of labor.

It is needless to say that neither concrete blocks nor hollow tile should be used if the walls of the power house are to be bearing walls.

Thin plastered walls on metal lath, which in turn is supported by a light steel framework, have been used in some small power houses. Such walls, however, are not recommended for this purpose on account of the difficulty, as already mentioned, of obtaining good plasterers, and the danger that unless

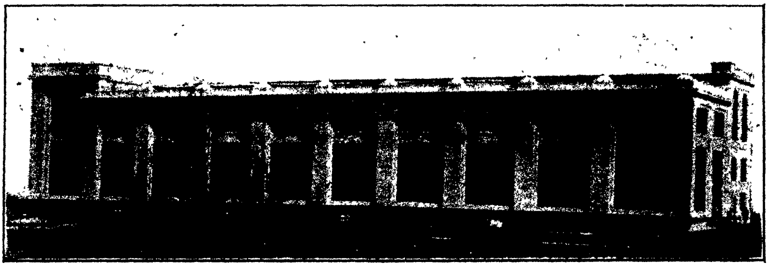


Fig. 366.—White River Station of the Puget Sound Power and Light Co. An Example of the Use of Concrete Walls.

such walls are very well built and braced they will crack more or less badly. Also, in severe climates such walls do not give sufficient protection against the cold weather.

308. Doors and Windows.—Sufficient ventilation is an important consideration in the design of any power house, and ample light should always be provided wherever possible. Particularly in warm weather it is necessary to have a steady and ample flow of fresh air into the building, and to provide exits for the hot air given off by the generating equipment.

It is never possible to have the power house comfortable in extremely hot weather, even in northern latitudes, and the greatest amount of window openings, consistent with proper architectural effect, should be provided. Too great a window area gives the effect of frailty, which is inconsistent with proper treatment for power-plant structures.

Steel and wood sash are used in power houses. The writer is of the opinion, however, that steel sash should always be used wherever possible. The use of steel sash reduces the fire risk to a minimum, and in most localities steel sash costs very little, if any, more than wood sash. Any of the standard makes of

steel sash can be used. There are several makes of cheap steel sash on the market, but these should not be considered in the design of a first-class power house.

There have been criticisms at times that it is difficult to make steel sash tight against wind and weather. However, if a good make of steel sash is used and the sash properly built into the walls, there is very little foundation for such complaints.

In building steel sash into the walls, if the walls are of brick, concrete blocks, or terra cotta blocks, the sash should be set up before the walls are built, in the same manner that wood frames are erected, and the walls then built around them. The writer is of the opinion that this method should also

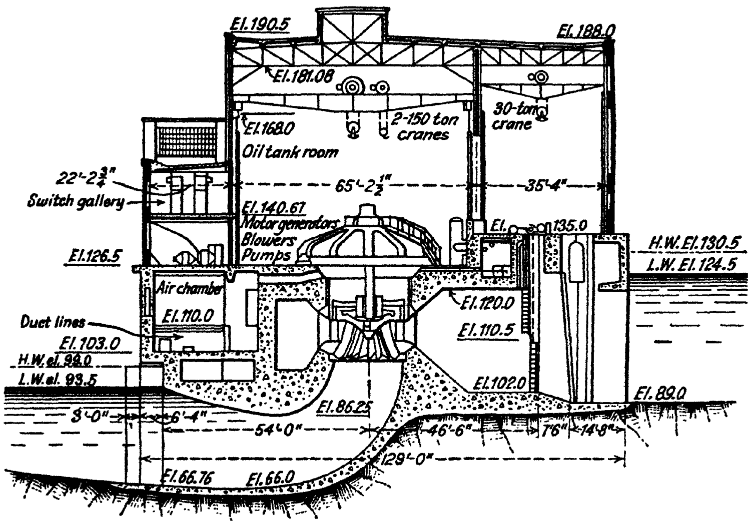


FIG. 367.—Cross Section of Cedars Rapids Power House Having a Concrete Superstructure.

be followed with reinforced concrete walls. This, however, complicates the form work somewhat, and often chases are left in the concrete walls and the sash set in place and plastered up after the forms are removed. If the work is done in this way, chases must be left on at least three sides of each window, with a groove on the fourth side, so that the sash can be slipped into place without chipping the concrete. When the sash is thus erected care must be taken to plaster thoroughly all cracks around the windows. This work must be done neatly or else the entire construction around the window will have a slovenly appearance. For some reason that the writer has never been able to discover, this work of setting and finishing around steel sash is one of the most difficult things to get a contractor or foreman to do properly. This work should be given special attention by the inspectors on the job.

If wood sash is used, the same remarks as to care in setting and making tight apply as for steel sash.

The maximum number of ventilating sections possible should be supplied if steel sash is used. If wood sash is used, it should be double-hung sash of not too large sections. In steel sash, ventilating sections should be placed as near the tops of the windows as possible.

In order to secure good ventilation at the top of the building, power houses are often provided with ventilators in the roof. There is always the danger, however, that leaks may occur in ventilators and water thus reach some of the electrical equipment. For this reason, ventilators, if used, should never be placed directly over electrical machinery.

A better plan for ventilation close under the roof is to provide sets of

Fig. 368.—Taylorville Power House of the Northern New York Utilities, Inc. An Example of Concrete Block Walls.

louvers on opposite sides of the building. These can be designed so as to come directly over windows and thus not affect the general appearance of the building. In fact, they can be treated with a panel effect, so that they are hardly noticeable. Either fixed or movable louvers may be used, depending on climatic conditions at the site.

A main entrance door, large enough to admit the largest piece of machinery, must be provided for every power house. This doorway should be placed in one end of the power house or in the side wall next to the end of the building. The landing bay is at this end of the power house, immediately in front of the doorway.

Either a sliding or a folding door may be used, each type having its advan-

tages. The sliding door is easier to handle, but is hard to make tight, which is a disadvantage in northern climates.

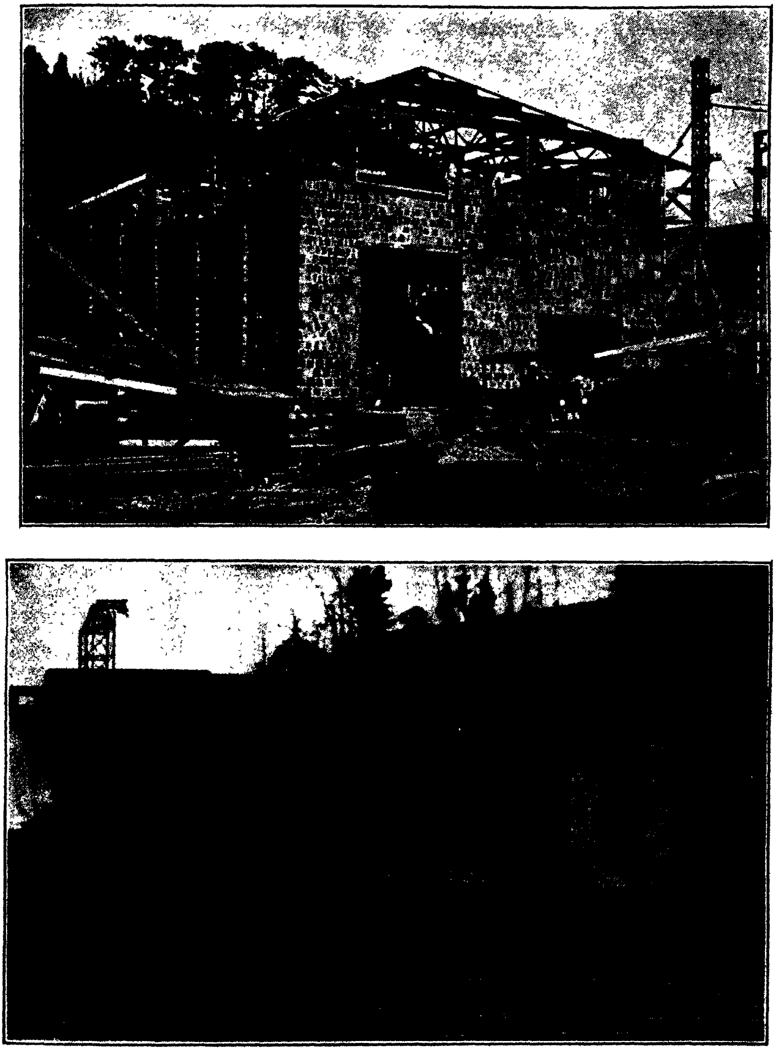


FIG. 369.—Sturgeon Pool Power House of the United Hudson Electric Corporation. During and After Construction. Terra-cotta Tile Walls Covered with Stucco.

Sometimes, on account of lack of space at each side of the doorway, it is impossible to use sliding doors in any event, and then folding doors must be used. These are generally built in 2, 3, or 4 leaves, and, as such doors run

from 14 to 18 ft. in height, and from 10 to 14 ft. or more in width, special provision must be made for attaching them to the door jambs.

The large entrance door should be provided with a wicket door unless it is advisable to have some smaller entrances in addition to the large one. Such

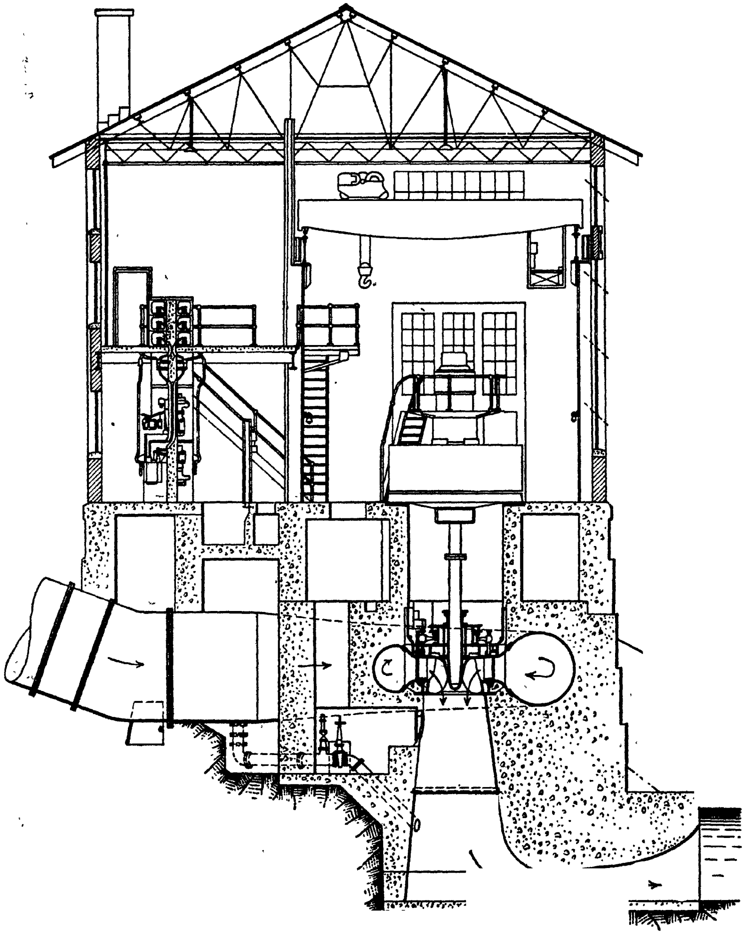


FIG. 370.—Cross Section of Sturgeon Pool Power House.

smaller entrances are often a convenience in giving direct access to the top of the head wall or the raking platform.

309. Floors.—The floors of all power houses should be of concrete. Those laid directly on earth should be not less than 6 in. thick and preferably laid on a cushion of sand or other suitable material.

When there is no basement or cable gallery to carry conduits, a distance of from 6 to 9 in. should be left, from the mass concrete in the foundation to the finished floor line, in which to bury the conduits. This space is necessary to take care of all conduits, wiring, pipes, etc. After all these are in place and everything tested, this space is filled in with concrete and the floor finished in the usual manner.

If the building has a basement, as in Fig. 370, the operating floor will be supported on reinforced concrete columns and beams or on structural steel work. In this case, in addition to the thickness of the slab necessary for strength, a 3 to 6 in. thick slab should be left on top to provide for small conduits, piping, etc., that have to be embedded in the floor. The larger conduits and main piping, however, will be located in the basement. After all conduits and piping are in place, this top slab is filled in and the floor finished.

The concrete used to fill in the top sections of the floor need not be as rich as that used for the reinforced parts of the floor. Whereas approximately a 1 : 2 : 4 concrete should always be used for the reinforced part of the floor, a 1 : 3 : 6 mixture, or even leaner, can be used for the top part of the floor in which conduits and pipes are embedded.

In the design of the floors, it is always a good plan to put a top coating on them about 1 to $1\frac{1}{2}$ in. thick, as part of the fill over the conduits. This top coating should be of much finer material than the other part of the floor, or it can be made of a mortar in which large-grained sand is used. It should be laid at the same time as the fill part of the floor, so that there will no danger of its separating from the lower layers of concrete. It also makes a harder wearing surface than the 1 : 3 : 6 fill would make.

Some kind of good floor hardener should always be used on power-house floors, in order to prevent "dusting." It is rarely economical or convenient to use an integral floor hardener in a power house, on account of the thick slab that forms the finished part of the floor. The writer has generally found it more convenient and more economical to apply a floor hardener after the surface of the floor has been finished.

The floors should be designed to support any machinery and materials that may be placed upon them during construction. As in most buildings, the floor is likely to be more heavily loaded during construction than subsequently.

It is rarely safe to design power-house floors for less than 300 lb. per square foot live load, and it is not often necessary to design them for more than 600 lb. per square foot live load. By careful planning it is sometimes possible, on account of the large size of machinery, to design the girders for the full live load determined upon, reduce this somewhat in the design of the beams, and still further reduce this load in the design of the concrete slabs. Such reductions in floor loads, however, must not be carried too far.

If there is an outdoor transformer station in connection with the power house, it is usual to provide facilities inside the power house for handling and repairing transformers if anything happens to them. As a basement is frequently built under the landing bay or part of it, a large hatchway can be provided through the main floor, opening into the basement. A transformer can then be brought into the power house, lifted by the crane, and lowered

through the hatchway until it rests on the basement floor. It can then be dismantled and the core removed by the crane with very little difficulty.

This hatch should be framed with structural steel, filled in with concrete, and provided with countersunk rings or holes so that it can be lifted up by the crane. The hatchway in the floor should be properly framed with angle curbs so that the hatch will fit neatly into it, and be supported by the necessary beams around the edge. The hatchway must be so located as to be commanded by the crane, so that the hatch cover can be lifted off by the crane, and the transformer lowered down into the basement.

310. Roofs.—Pitched roofs, similar to the type shown in Fig. 370, are sometimes used with varying degrees of angles, particularly in climates with exceptionally heavy snowfall. However, the flat roof, shown in most of the illustrations, with parapets or with simple overhanging eaves, is now the type generally adopted. Fig. 371 shows details of a parapet and of an overhanging roof.

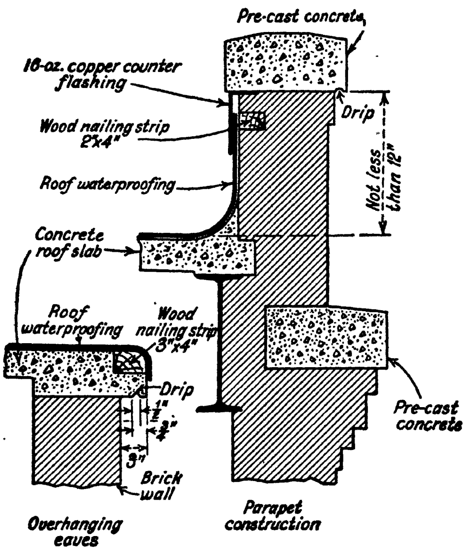


Fig. 371.

The flat roof with overhanging eaves and only enough pitch to give proper drainage is the cheapest type. The minimum slope which should be provided is about $\frac{1}{4}$ in. to the foot. The objection to overhanging eaves in cold climates is the fact that icicles will form along the eaves and may grow to enormous size. On account of the position of the windows in power houses, such ice formations are difficult to dislodge, and this is the main reason why parapets should be used in conjunction with flat roofs in such latitudes.

One objection raised to the parapet type of roof is that snow will accumulate within the parapets, thus bringing an excessive load on the roof. On very rare occasions this might happen, but the writer has watched many roofs of this type in northern climates and has never seen it occur. A wet, heavy snow that would stay on such a roof is generally of very short duration. Any other snow that will not stick as it falls invariably blows off the roof, except for small patches in the corners.

Formerly wood was used to a considerable extent for the roofs of power houses. Its advantages are that it is cheap and easily applied. Its disadvantages are that it is not fireproof and has a short life when compared with other types of roofs. It has no place in a modern power house.

The roof is generally built of simple reinforced concrete slabs. These should in no case be less than 3 in. thick and preferably 4 in. as a minimum.

Many other types of fireproof roofs have been used, some of which are patented articles. These include Hy-rib and similar types of reinforced concrete slabs, gypsum roofs, cast-in-place or pre-cast slabs, etc. The simple concrete roof reinforced by steel mesh or bars seems the most suitable except for special conditions.

Whatever type of roof is used, it must be protected by a waterproof covering. The one that is most generally used and is probably the most satisfactory is the 3, 4 or 5-ply built-up roofing finished with a mopping of tar or asphalt and without any slag or gravel.

The 3 and 4-ply roofs are cheaper than the 5-ply, and some manufacturers claim that their 3-ply roof is equal to a regular 4-ply. The writer, however, does not advocate a 3-ply roof, but prefers the 4-ply for general conditions with a ten-year guaranty.

The disadvantage of slag or gravel over a power-house roof is due to the fact that such plants are often far removed from centers of population. If a leak develops there is not time to send for the roofer, even if there is a ten- or twenty-year guarantee on the roof. The operator must make repairs himself, generally by mopping hot materials over the part of the roof where the leak has occurred. It simply complicates matters if he has to remove slag or gravel before making such repairs. Any of the standard types of built-up roofing are satisfactory, but a cheap roof or one that does not have a final mopping over its entire surface should not be considered. Competent roofers should always be employed to apply the roofing, and a ten-year guaranty insisted upon. Such a guaranty tends to eliminate poor workmanship and materials, if it accomplishes nothing else.

If the power house is built with a parapet, copper or lead counter flashing should always be used. The roofing material should be carried up on the walls under the counter flashing. This should be built into a joint in the brick work or into a reglet provided for it in the parapet wall and carefully turned down over the roofing material so as to make a tight and weatherproof joint. Fig. 371 shows this construction.

311. Stairs and Railings.—Stairs should always be of steel or concrete. They can be built with channel iron strings, open risers, and checkered steel plate, cast-iron, or bar treads. Sheet-steel treads, filled with concrete, also make an excellent stair.

Stairways for power houses are seldom built on a flatter slope than 45 degrees, although steeper ones are permissible if room is not available, particularly if they are seldom used. They should always be provided with hand rails. If the stairway has a flatter slope than about 45 degrees, and one side is against the wall, a hand rail on the outside only is required. If, however, it is steeper than about 45 degrees, or if it is not built against a wall, a hand rail on both sides is required.

For 45-degree slopes, the treads and risers are usually 8½ to 9 in. For flatter slopes, the treads remain the same and the risers are reduced in height

to correspond to the slope. For steeper slopes, the risers may be increased to 9½ or 10 in. and the treads reduced accordingly.

Where concrete stairs are used, the surfaces should always be finished rough, with a wood float, or safety treads used.

There are several excellent types of all-pressed-steel stairs on the market and these can often be used economically.

Railings should always be used wherever possible, to protect openings, stair wells, platforms, etc., both inside and outside the power house. A double rail, the first 22 in. above the floor, and the second 20 in. above the first, making a railing 3 ft. 6 in. high, should be used. One and one-quarter-inch black iron pipe is amply large for all railings. Standards should be well anchored to concrete by means of flanges with not less than 4 bolts, ½ by 6 in. long. Standards should be not more than 8 ft. 0 in. apart. Floor flanges should be standard companion flanges, 4½ in. diameter.

All stair wells and edges of galleries should be provided with a concrete toe board 6 in. high, as shown in Fig. 372. If a railing is placed on top of this toe board, it need only be 3 ft. 0 in. high above the top of the toe board.

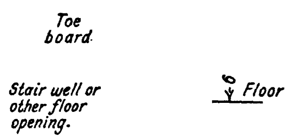


Fig. 372.

Care should be used always to fasten standards securely. There is nothing so unsightly as a loose and rattling railing, and such a railing is often a source of danger.

312. Water Supply and Drainage.—The water supply for the bearing and transformer cooling systems and for general use is obtained from the nearest source of continuous supply. For high-head developments the supply is from the penstocks, while for low-head plants it is taken directly from the intake.

In the case of high-head developments, a header of ample size should be connected to each penstock and provided with the necessary valves so that water will be available even if all the penstocks but one are unwatered. For the same reason, the inlet of the water supply at the intakes of low-head developments should be in an independent bay; or else there should be one in each main bay and all of them should be interconnected by a header. A connection to a city water supply, if convenient, might be desirable as an added safeguard; and, if the river water is extremely silty at times or otherwise unsuitable, the use of the city supply may obviate the necessity for an expensive filter plant, which it has been found necessary to install in some instances.

Usually the turbine manufacturer provides an independent source of supply for the bearing cooling system of each turbine, direct from the wheel casing of the respective units.

In a low-head plant, it is often necessary to provide pumps for the water supply, if the head is not sufficient for all purposes. If there is no alternate water supply available, such pumps must be installed in duplicate to insure an

uninterrupted flow of water if one pump is out for repairs. The suction line to such pumps should have a foot valve and strainer on the intake end.

Water supply should be provided at convenient points around the power house. Such supply pipes should not be less than 2 in.; if desired, they can be bushed down to $\frac{3}{4}$ in. and ordinary angle valves provided from which to fill pails. Such supplies should not be placed near electrical equipment, and it is convenient to have floor drains close by, to take care of any drip or splash from the valves.

Fire connections are not required in modern fireproof power houses, reliance being placed on the use of chemical fire extinguishers.

All piping installed for water supply should be galvanized genuine wrought-iron pipe, with galvanized malleable or cast-iron fittings. The first cost of wrought-iron pipe is higher than that of steel pipe, but it will outlast steel pipe, and in the end is more economical.

Provision should be made to strain the water supply either by screens at the intake or strainers in the supply line from the penstock connections. The screens or strainers must have adequate provision for cleaning without shutting off the supply of water.

A lavatory and toilet must be installed in every power house, and if the building is large enough, a slop sink should also be provided. The soil pipe can be led into the draft tube or tail race at any convenient point. All soil pipe should be standard-weight cast-iron soil pipe and fittings, unless the development is in a community that requires a heavier weight of pipe. Steel or wrought-iron pipe should never be used for soil pipe.

If a parapet type of roof is selected, roof drains of the most approved make should be used, as leaks are a great menace to electrical equipment. All leaders should be galvanized-steel or wrought-iron pipe. Leaders should drain into the draft tube or tail race.

In cold climates all piping must be kept on the inside of the building, and as far away from the outside surfaces of walls as possible, in order to prevent freezing. If possible, piping should not be buried in mass concrete unless it is more than 2 ft. 0 in. from the outside surface. Even then, during times of continued cold weather, any such pipe may freeze. All pipes running into the tail race should preferably be extended well under low tail-water, as otherwise the splash of water may plug the outlet of the pipe with ice.

All drains should be run in as straight lines as possible, to avoid plugging, and clean-outs should be provided wherever necessary, to facilitate clearing any drain pipe that becomes plugged.

All floors should be provided with drains to carry off water when the floors are washed. Such drains should be placed away from electrical equipment, so that water will drain away from all apparatus.

313. Power-house Crane.—The traveling crane is generally the largest and heaviest piece of auxiliary machinery in the power house. Its capacity, and therefore its weight, is determined by the heaviest load that it must handle. However, it handles such a load only at rare intervals, when the runner and its attached rotor are being set, and this is done with extreme care. The crane in a hydro-electric power house is never subjected to the severe service to which

a foundry crane or machine shop crane is subjected, and therefore a lighter type of crane can be installed than would be advisable for heavier service.

A one-trolley or a two-trolley crane can be used. Figure 373 shows a two-trolley crane. The one-trolley crane is considerably cheaper than the two trolley; but, on the other hand, it requires a higher power house to give sufficient clearance. With two trolleys, the shaft of either the rotor or the runner can be hoisted up so as to pass up between the trolleys and the crane bridge girders, thus materially reducing the height of the power house necessary to obtain clearances (see Fig. 373). This cannot be done with a single trolley. On the other hand, it is much easier to adjust the rotor and runner with a single trolley, as there is only the one hoist to manipulate. If the height of the power house is to be determined by the clearance necessary to carry the rotor or the runner, with its shaft, over the other units, then the two-trolley crane will probably be the most economical. If, however, for some other reason, the power house has to be built higher than otherwise would be necessary to obtain the above-mentioned clearance, and there is height enough for a one-trolley crane, then this installation is unquestionably the cheaper.

In setting the runner and rotor of a vertical unit with the crane, the following procedure is followed:

The runner is lowered into place. In the Francis type of runners, the turbine casing has a ring on which the runner can rest. In the propeller type of runner, it is generally necessary to use blocking on which to rest the runner, although the turbine manufacturers sometimes provide special lugs which are bolted to the casing, and on which the runner can rest.

After the generator stator is erected, the rotor is lowered into position and leveled up temporarily on blocking or on the generator brakes. This generally leaves from 1 to $1\frac{1}{2}$ in. clearance between the generator-shaft coupling and the turbine-shaft coupling. The coupling bolts are then drawn up and the runner raised to its final position. The thrust bearing is then installed and adjusted and the temporary support of the rotor is removed.

A clearance diagram similar to that shown in Fig. 373 should always be made and submitted for approval to the crane, turbine, and generator manufacturers.

314. Telephone Booth.—The telephone booth should be enclosed with terra cotta, brick, or concrete walls and ceiling, and provided with a good single door, or preferably a double door, to keep out as much noise as possible. A false wooden floor, supported on glass insulators, should also be provided, to reduce as much as possible the chance of a heavy shock to anyone using the telephone.

315. Heating.—In cold climates, the question of heating a power house is one that must be given consideration. At ordinary temperatures, down to freezing, enough heat is given off by the generators to keep the body of the building comfortably warm. But at lower temperatures, or in a large power house where a number of the units may be shut down at the same time, other means must be provided for supplying enough heat to keep at least a part of the power house, where the operator is stationed, reasonably warm (68°). For this reason, in all power houses in cold latitudes, one or more chimneys, with proper flues, not less than 8 by 8 in., should be provided. These chim-

neys should be located at convenient points, so that stove pipes can be easily connected to them. All chimneys should have terra cotta flue lining, surrounded with 8 in. of brick.

In some cases, the switchboard is in a separate room or is so located that

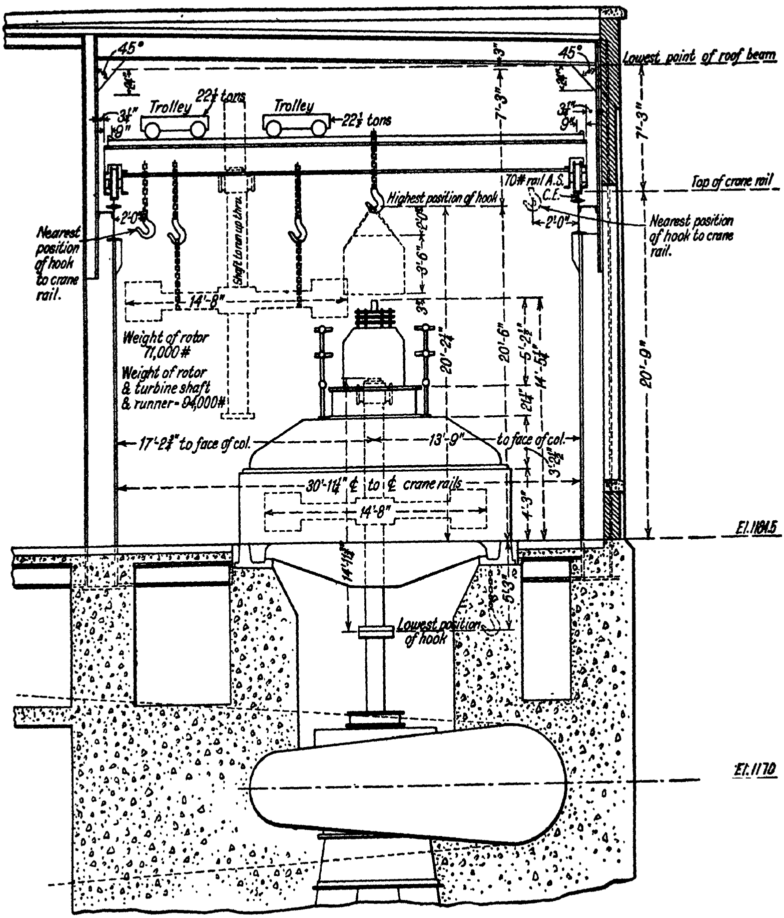


Fig. 373.—Typical Crane Clearance Diagram.

it can be enclosed in a glass partition, and this enclosure heated with electric heaters or small stoves. Another method is to provide a small heated portable enclosure in front of the switchboard, in which the operator has his desk and chair.

CHAPTER XXVIII

HYDRAULIC TURBINES

By William M. White

316. Introduction.—Power may be developed from water by three fundamental processes: by weight alone; by pressure alone; or by velocity alone. The first method, that of developing power by weight alone, may be illustrated by the overshot wheel which was extensively used in the early days for driving saw mills and grist mills. The pressure means may be well illustrated by considering a reciprocating engine being driven by water pressure. Here the pressure alone is used to develop the power. The third means, by velocity, may be well illustrated by the Hurdy-Gurdy, a jet-driven paddle wheel used by the early California miners where high heads and small amounts of water were available. The invention of Lester Pelton greatly improved the efficiency of this type of water wheel.

At the present time, only the latter two means are extensively employed for efficiently converting the energy of water into power. The reaction, or Francis, runner, as shown in Fig. 381, is a combination of the pressure and velocity types of machine. The impulse, or Pelton, turbine, shown in Fig. 387, is a purely velocity machine. The Propeller, or Nagler, type wheel, illustrated in Fig. 376, is also a combination of the pressure and velocity machines, although it employs the velocity principle to a greater extent than the reaction type wheel.

317. Types of Hydraulic Turbine Machinery.—There are five principal types of machinery using these three methods for efficiently developing power from water. There are several other less important types of machinery which are in more or less common use, but the following five types are the most efficient and are used in the majority of hydraulic installations:

The first of these is the open-flume turbine shown in Fig. 375, using either a Francis or a propeller type runner; second, the concrete spiral-cased turbine shown in Figs. 376 and 377, using either a Francis or a propeller type runner; third, the plate steel spiral-cased turbine illustrated in Fig. 381; fourth, the cast-iron, or cast-steel spiral-cased turbine shown in Figs. 383 and 387; and fifth, the impulse turbine shown in Figs. 388 and 389. Fig. 374 shows the limits of head and capacity within which these varous types are ordinarily used. The vertical scale is horse power per unit; the horizontal scale is effective head. The different areas are labeled with the name of the one or more types of units which are commonly used to meet these conditions of power and head.

The open-flume turbine as shown in Fig. 375, is for low-head developments. Sizes up to 1000 h.p. at 10-ft. head, and 2500 h.p. at 25-ft. head are about as large as turbines of this type can be economically constructed. The majority of open-flume turbines are of the vertical-shaft type, as with this arrangment it is possible to get greater efficiency. There are a large number of horizontal open-flume turbines of small and medium capacities in operation, but the trend of new developments is entirely toward the vertical setting, except in the case of extremely large-sized units, where the matter of efficiency is overbalanced by the cost of installation. In this case horizontal units of the multiple-runner type are resorted to in order to obtain greater power at higher speeds. However, it is usually more economical to resort to the concrete spiral-cased setting for the large capacities.

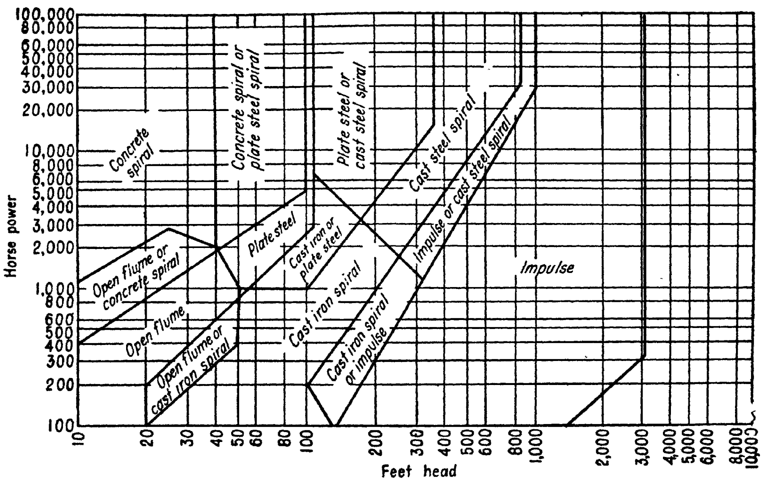


FIG. 374.—Curves Showing Capacities and Heads for Which Various Types of Turbines are Suited.

The concrete spiral-cased setting, as shown in Figs. 376 to 379 is used principally for capacities above 400 h.p. at 10-ft. head and above 5000 h.p. at 100-ft. head. Concrete spiral-cased units are rarely used above 100-ft. head, as the problem of reinforcing the concrete makes them more expensive and less reliable than the metal or cast type of casing. Concrete spiral-cased turbines are invariably of the single-runner vertical-shaft type. Scarcely any installations have been made with horizontal shaft.

Plate-steel spiral-cased turbines, as illustrated in Figs. 380, 381 and 382, are suitable for heads above 40 ft. and up to 375 ft., although for heads higher than 375 ft. the cast type of casing is usually employed. Plate-steel spiral-cased turbines are usually of the vertical-shaft single-runner type because it is possible to obtain greater efficiency with a vertical setting, although there are some installations of horizontal-shaft units of moderate and small capacities.

Cast spiral casings, Figs. 383 to 386, are of two types, cast-iron and cast-steel. Cast-iron casings are used for moderate and small-sized units under medium heads, but cast-steel casings should be used for units of large capacities, even under medium heads, and for all capacities under high heads. The reason for this is that cast iron is not a reliable material when high tensile stresses are to be carried, such as are caused by internal water pressure when used for large capacities or for high heads. Cast steel is a much more ductile material and its tensile properties are practically equal to its compressive qualities.

Cast-spiral casings are used for small units under low heads and for large capacities above this up to 1000-ft. head. For heads above 1000 ft. the cast-casing type of turbines is not usually employed because the Francis type of runner gives too high a speed for safe and economical generator construction. For this reason, impulse turbines are used under heads higher than this. Cast spiral-cased turbines are of the single-runner vertical-shaft type for large sizes under all heads and for small capacities under medium heads. Under some conditions the horizontal setting is used, two turbines being used to drive one generator as shown in Figs. 386 and 387. The advantages of the twin horziontal spiral-cased turbine are that greater generator speed may be obtained and that the power house may be greatly simplified in design, both the generator and the turbine being on one floor. While the efficiency of the horizontal high-head Francis turbine is not quite as great as that of the vertical setting, the difference is much less for this type of runner than for the type used with the lower-head turbine.

Impulse turbines, Figs. 388 and 389, are used for heads above 850 ft. and capacities of 30,000 h.p. They are also used on smaller units down to 200 h.p. at 100-ft. head. Impulse turbines have been used successfully up to heads of nearly 3000 ft., but at the present time it does not seem economical to develop plants for heads higher than 3000 ft. because of the problem of designing and building penstocks and pipes to carry any great quantity of water under this pressure. For this reason, where heads above 3000 ft. are available they are usually split up into two or more plants. The highest-head impulse plant in America is the Bucks Creek Plant of the Feather River Power Company, which reaches a maximum of 2575 ft. static. There are impulse plants in Europe which reach 5000 ft. Impulse turbines are usually of the horizontal-shaft type, either one or two sets of buckets and disks being used to drive each generator. A construction which has worked out very economically and efficiently in this country is the double overhung type shown in Fig. 389, with one set of buckets and disks overhung from each end of the extended generator shaft, the entire weight of the generator rotor and the two impulse wheels being carried by the two generator bearings. These impulse wheels use only one jet on each wheel. As larger capacities under medium heads are required, it is probable that vertical-shaft impulse wheels, as illustrated in Fig. 390, with two or more jets per unit, will come into more common use, although at the present time vertical-shaft impulse wheels are few.

These types of hydraulic turbines are the principal and most efficient ones. The limits given above are the ordinary limits within which each of these types

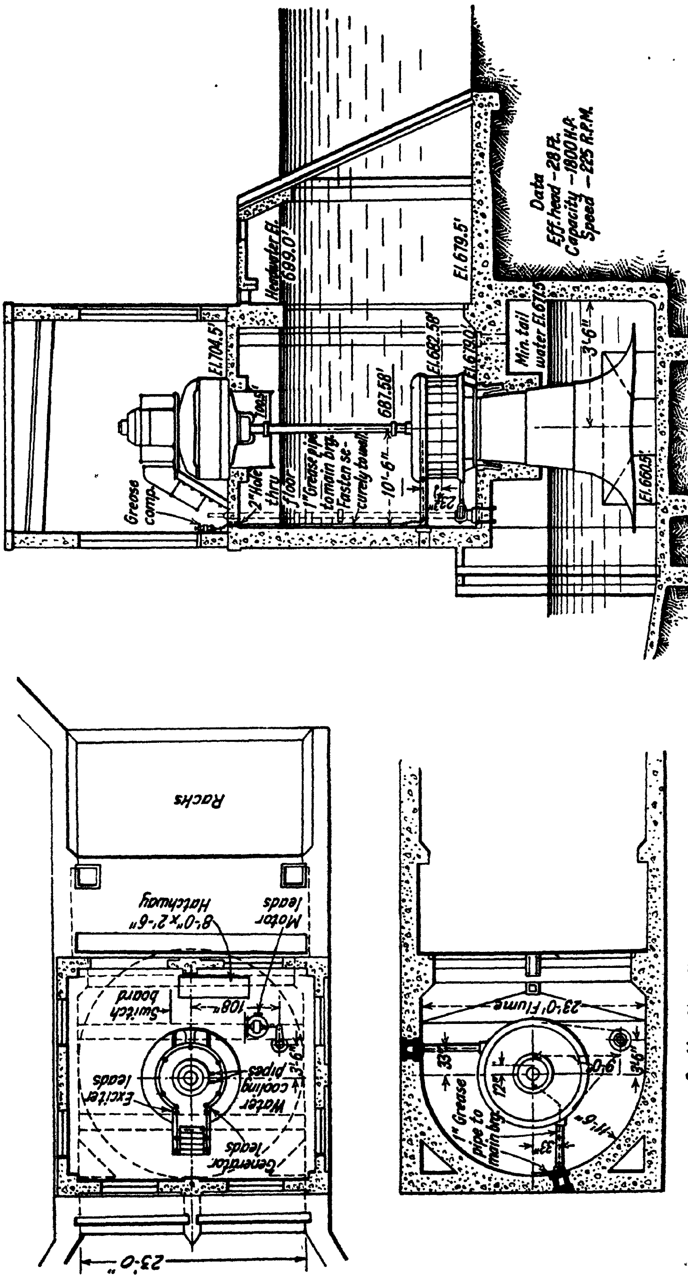


FIG. 375.—Single Runner Vertical Open Flume Setting for Economical Construction and Reliable and Efficient Operation. Plate steel draft tube simplifies form work. Gate mechanism located on discharge ring, gate shaft extended up to generator floor for connection to governor.

is generally used, although there are some special cases in which it has been found that another type would result in a more economical development.

Figure 375 shows a good design of single-runner vertical-shaft open-flume setting. The water enters through trash racks into an open flume which may or may not have a circular form on the downstream side. The circular form of flume, while being slightly more expensive to build, has certain advantages in reducing disturbed flow. This type of turbine is usually set on the floor of the flume and the water is discharged downward through either a concentric or an elbow-type draft tube. The plate-steel concentric type of draft tube

FIG. 376.—Vertical Concrete Spiral Cased (Syphon Setting) Turbine with Propeller Runner 800 h.p.—8-ft. Head, 80 r.p.m. Dixon development showing column type speed ring, plate steel pit liner and concentric plate steel draft tube with cone center. (I. P. Morris.)

shown here has many advantages. It is not expensive, it saves a great deal of time and form work in the concrete construction, and its efficiency is equal or better than that of the other types. The generator is shown mounted on the generator floor of the power house above the maximum head-water. This does away with a bulkhead and stuffing box around both the main and regulating shaft with the resultant disadvantages such as leakage, friction, and uplift on the generator floor, which would occur if the generator floor was below the head-water level.

Stop-log slots are provided behind the racks, so that the water may be shut off from the flume for inspection and other purposes. The governor

which operates the gate mechanism of the turbine is located on the generator floor, the regulating shaft extending down into the flume where it is connected

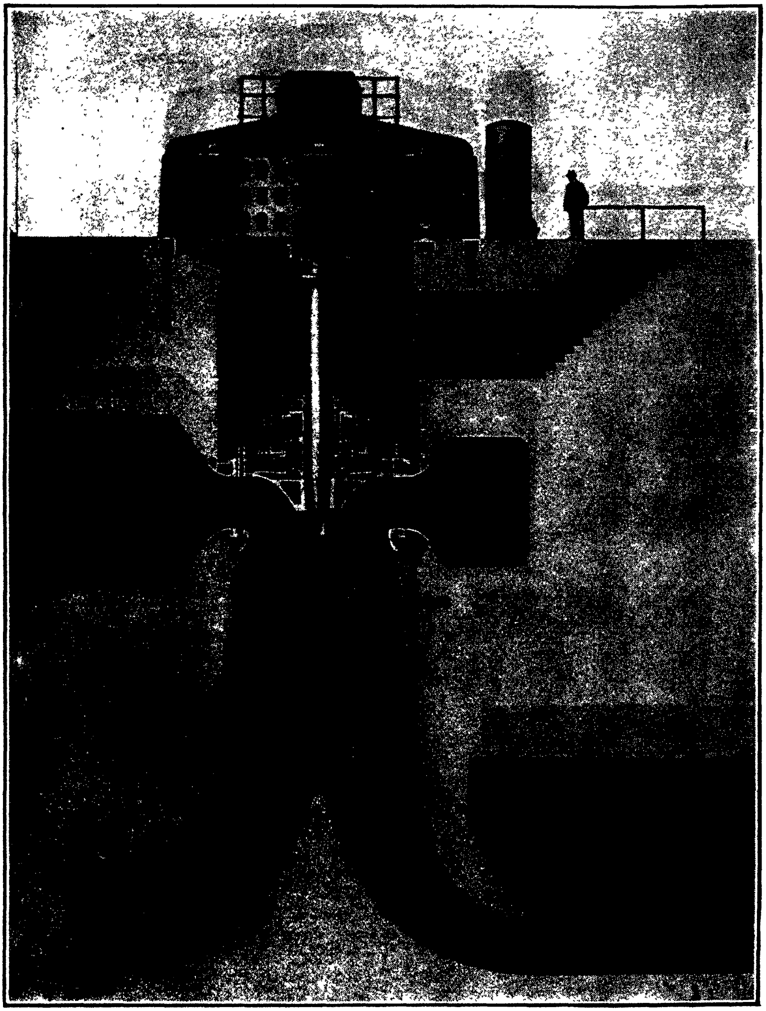


Fig. 377.—Vertical Concrete Spiral Cased Turbine with Francis Runner, 30,000 h.p. 95-ft. Head, 100 r.p.m. Muscle Shoals Development, showing water-lubricated bearing, cast-iron pit liner supporting servo motors. (I. P. Morris).

to the shifting ring through two operating rods. The turbine cover is supported or braced from two sides of the flume, in order to stiffen it and also to resist the backward forces exerted by the water when passing between the

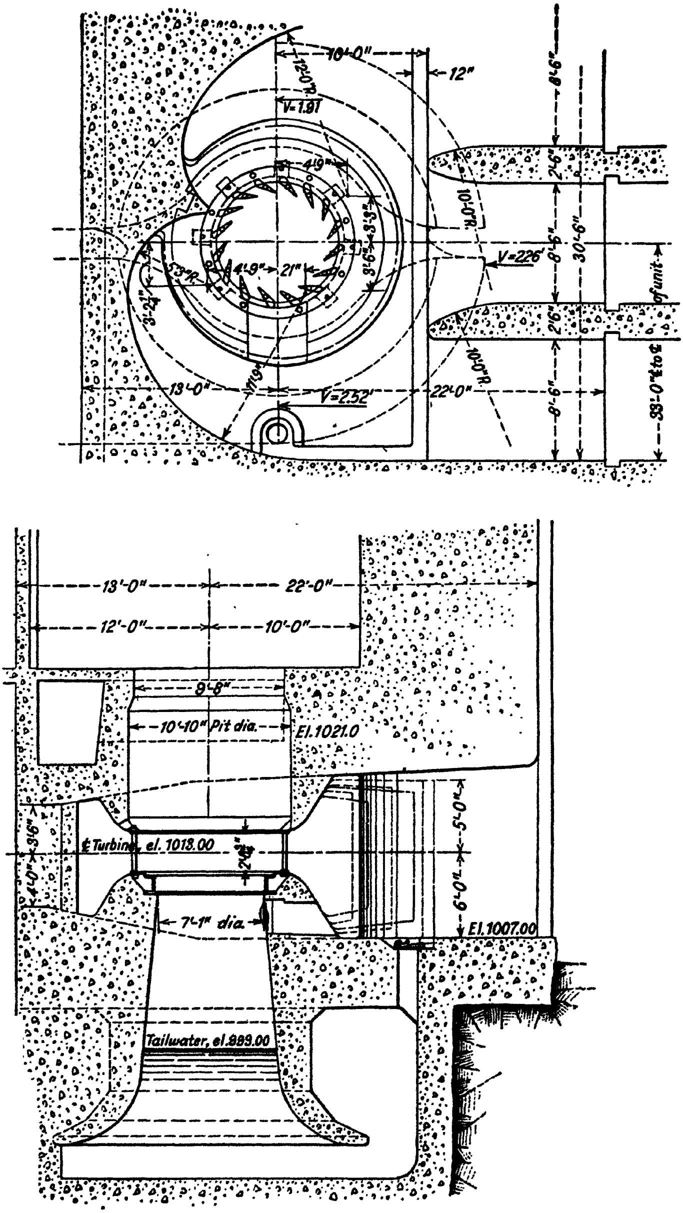


FIG. 378.—Plan View and Section of Concrete Spiral Casing, Having Nose at Rear, showing Staybold Type Speed Ring, Drain Valve, Draft Tube Liner with Inspection Manhole, and Concentric Concrete Draft Tube.

guide vanes. The water velocities entering the flume should be low, from 2 to 3 ft. per second maximum, as high velocities in unguided water passages, which exist in open flumes, frequently set up disturbances which seriously affect the performance of this type of unit.

Figure 376 is a sectional elevation of a good example of single-runner vertical-shaft concrete spiral-cased turbine of the propeller type, a plan view of this type of casing being shown in Fig. 378. The water enters the concrete casing through suitable trash racks, headgates similar to those used with the open-flume setting being provided to shut off the water for inspection. The water is led to all sides of the turbine, the design of the casing being such as to give, as nearly as possible, uniform velocities in all parts.

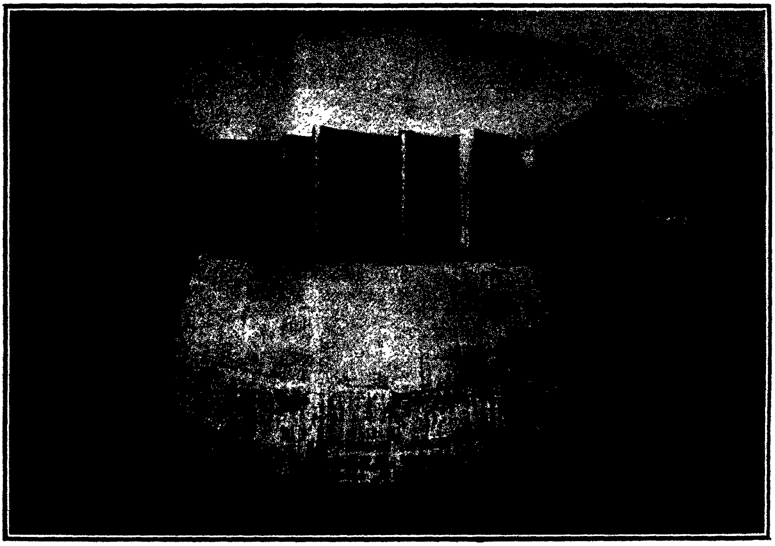


FIG. 379.—View Taken Inside of Concrete Spiral Cased Unit, 2500 h.p. 21-ft. Head, 138½ r.p.m. Wausau Sulphate and Fibre Company. Showing prints left by form lumber, cast-iron columns of speed ring and plate steel guide vanes.

The draft tube shown here is of the concentric or spreading type, constructed of plate steel. The generator is shown mounted on a short supporting barrel, and is reached by a short stairway, access to the turbine cover plate being had through an arch and stairs leading down into the turbine pit, above the small end of the spiral. The governor is located on the main floor, and the servo-motors or regulating cylinders for controlling the position of the gates are located in the turbine pit. For large-sized units where the governor capacity is above 25,000 or 30,000 ft.-lb., the regulating cylinders are located in the turbine pit, connection being made directly to the shifting ring; but units requiring smaller governors are usually equipped with a vertical regulating shaft similar to that shown in Fig. 375.

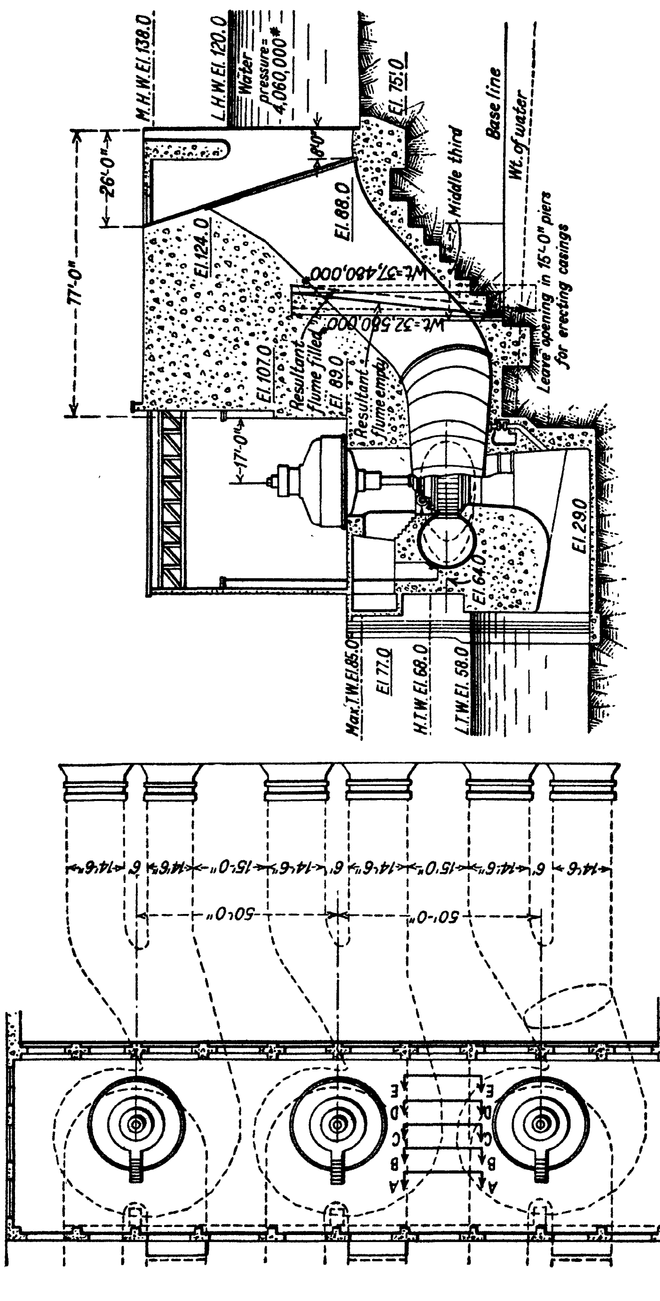


FIG. 380.—Power House Drawing for Plate-steel Spiral-cased Units, 20,000 h.p., 70-ft. Head, 100 r.p.m. West Kootenay Power and Light Company, showing intakes through dam, elbow-type draft tubes and special construction of generator air vents because of high floods in tail-water. Lower Bonnington Plant of West Kootenay Power and Light Co., British Columbia.

Figure 377 shows the arrangement of a concrete spiral-cased Francis turbine, rated 30,000 h.p under 95-ft. head. This is one of the largest as well as the highest-head units for concrete spiral-cased setting and was installed in the Muscle Shoals plant of the U. S. Government. The runner is of cast iron

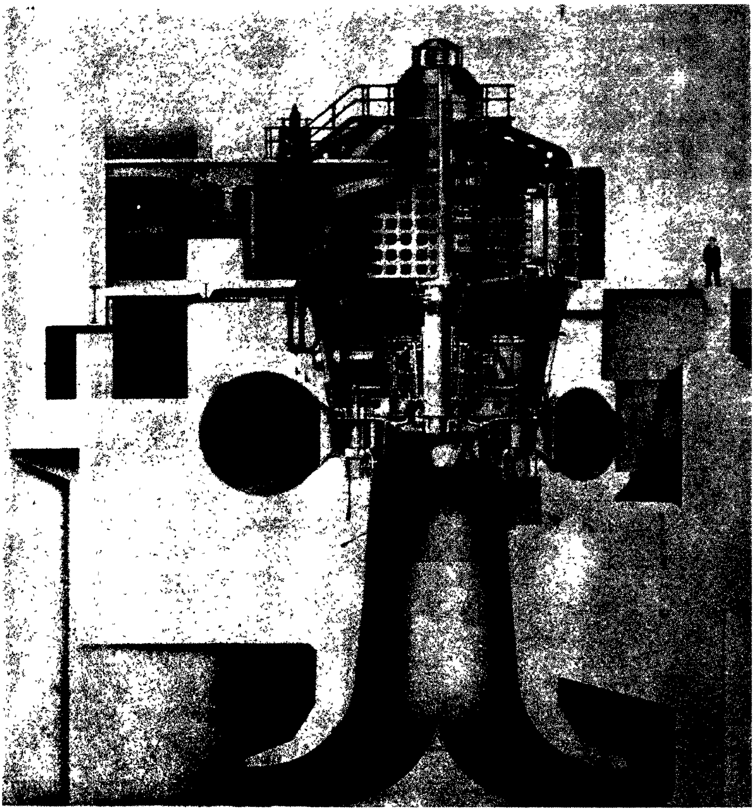


Fig. 381.—Section through 70,000 h.p. 213-ft. Head Plate Steel-cased Turbine and 65,000 K.V.A. 13,000-volt 3-phase 25-cycle 107 r.p.m. Generator, Niagara Falls Power Company Unit No. 21. Concentric draft tube of concrete with low cone center, cast-steel runner bolted to flange on shaft, adjustable lignum vitæ bearing, steel sleeve on shaft, cast-steel speed ring, cast-iron pit ring and supporting barrel carrying weight of generator, direct-connected governor flyballs, regulating cylinders bolted to pit ring and connected directly to opposite sides of shifting ring. Kingsbury thrust bearing above generator, turbine bearing serves as lower generator bearing, making two bearing unit. (Allis-Chalmers Manufacturing Co.)

in one piece, keyed to the tapered main shaft. The speed ring, which supports the weight of the parts above the casing, is cast with ribs and flanges integral, and a cast-iron pit liner, to which the regulating cylinders are bolted, lines the turbine pit and helps transmit the weight of generator and concrete

to the speed ring. A water-lubricated lignum-vitae main steady bearing supports the shaft, which runs at 100 r.p.m.

Figure 379 is a photograph taken inside of the concrete spiral casing for a 2500-h.p., 21-ft. head vertical turbine. Figure 378 shows the plan view of such a casing. This shows quite clearly the arrangement of the columns of the speed rings, which carry the weight, and the guide vanes or wicket gates which control the amount of water entering the turbine. Concrete spiral-cased turbines are usually built into and form a part of the dam, so that the water passage from the intake through the dam to the unit is relatively short.

Figure 380 shows the general arrangement of a power house containing three single-runner vertical-shaft plate-steel spiral-cased turbines for about 70-ft. head. These turbines are located adjacent to the dam, so that the water is led through openings in the dam to the inlet of the spiral casing. The draft tube shown on these units is of the elbow type which, while not having all the advantages of the concentric type of draft tube, can be squeezed into a smaller space, thus saving considerably on the total length of the power house. The draft tube is lined in the upper part with a cast-iron base ring and a plate-steel liner. This is desirable because there is danger of washing away the concrete when the water is traveling at a velocity greater than 10 to 12 ft. per second, especially when whirls, eddies, and vacuum are present.

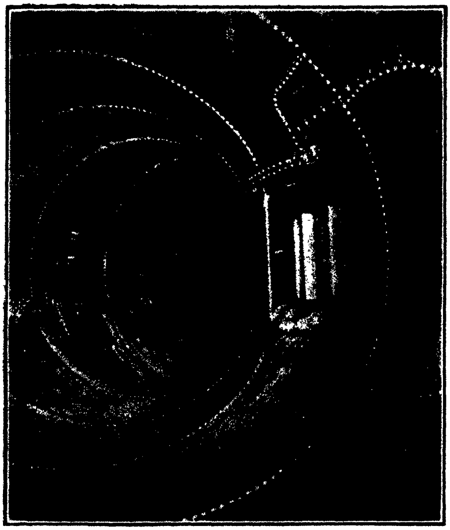


Fig. 382.—Photo Looking into 10½-ft. Inlet Diameter Plate-steel Spiral Casing of 2600 h.p. 40-ft. Head, 150 r.p.m. Ludlow Manufacturing Associates showing speed ring with guide vanes in closed position. Single-riveted casing joints with double riveting at junction with cast-steel speed ring (Allis Chalmers Manufacturing Co.).

Figure 381 is a section through the 70,000-h.p. Unit No. 21 in the plant of the Niagara Falls Power Company. At the present time this is one of the largest-capacity turbines in the world. This figure shows clearly the plate-steel spiral casing, the cast-steel speed ring which holds the edges of the casing together, and the guide vanes and operating mechanism which control the flow of water to the runner. The draft tube shown here is of the concentric concrete type with a plate-steel liner in the upper part. In this case the weight of the generator is carried on a cast-iron supporting barrel, which transmits the load through the speed ring of the turbine to the foundation. This unit has only two main bearings, the one located on the cover plate of the turbine,

the other just below the thrust bearing on the bridge above the generator. Figure 382 is a photograph taken inside of a plate-steel spiral casing and shows the method of connecting the separate plates which make up the casing, as well as the joint between the plate-steel casing and the speed ring. The first opening through the speed ring where the water enters the guide vanes is clearly shown. While round-headed rivets are here shown holding the plates together, more recent practice, especially on the higher-head plate-steel casings where the water velocities are greater, has been to use the flat-headed countersunk rivets, which offer very little obstruction to the flow. While some plate-steel spiral-cased turbines are located adjacent to the dam, many turbines of this type are at a considerable distance from the dam, the water being led down to the units through wood-stave, or plate-steel penstocks. When penstocks are used a valve of the butterfly, gate, or plunger type is frequently used to shut off the water at the inlet of the spiral casing. There are, however, some installations where no valves are used, the water being shut off by headgates located at the upper end of the penstocks, these headgates being similar to the type used with the open-flume and concrete spiral-cased turbines.

Figure 383 shows the power-house arrangement for two single-runner vertical-shaft cast-steel spiral-cased turbines. These units are equipped with hydraulically operated butterfly valves at the inlet to the casing, and governor-operated pressure regulators or relief valves are used to prevent serious pressure rises from occurring, their action being described fully in Sec. 337. The governors are of the actuator type with the regulating cylinders located near the cover plate and connected directly to the shifting ring. The generators are mounted on a separate floor. There are four floors in this type of power house, the lowest floor being below the turbine around the draft tube, the second floor being at the level of the turbine cover plate, the third floor at the generator base line, and the fourth floor on a level with the switchboard and the top of the generator. The draft tube is of the elbow type.

Figure 384 shows a section through Units Nos. 19 and 20 in the plant of the Niagara Falls Power Company. These are also rated 70,000 h.p. and are of the cast-steel spiral-cased type, a close view of the cast casing being shown in Fig. 385. Aside from the casing, the design of this unit is essentially similar to that of Unit No. 21 shown in Fig. 381, and although the two units are made by different manufacturers, the cast-steel runners and lignum-vitae guide bearings are interchangeable. In Fig. 384 is shown a concentric or plunger-type valve which is located at the inlet of the spiral casing and is used to shut the water off from the turbine.

Figure 386 shows a double overhung unit where two cast-steel spiral-cased turbines are used to drive one generator, the weight of the two runners and the generator rotor being carried on the generator bearings. These units may also be equipped with pressure regulators, and each unit may have its separate governor and governor system, so that either turbine may be run independently. This permits of much more efficient operation below half-load. Fig. 387 is a section through one of these horizontal turbines, showing the arrangement of the various parts.

Cast spiral-cased turbines usually have the water brought in through long penstocks, which makes it very desirable to provide some means for shutting off the water at the inlet of the casing.

Figure 388 shows a single overhung horizontal impulse turbine having one jet operating on the buckets and disk. The buckets and disk are mounted

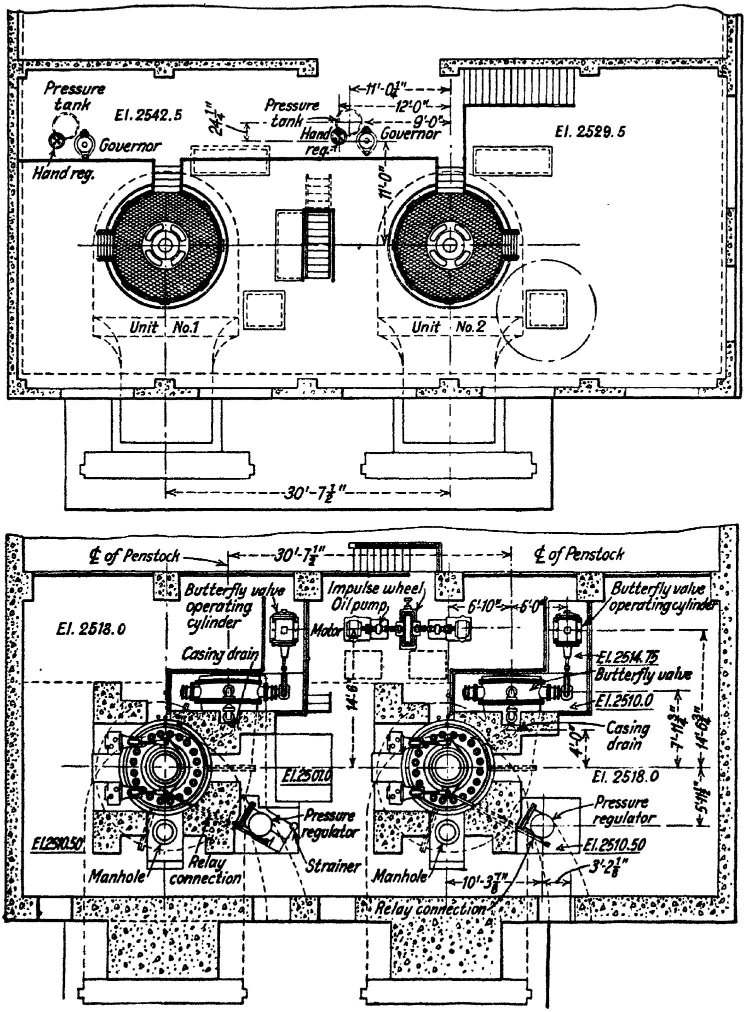
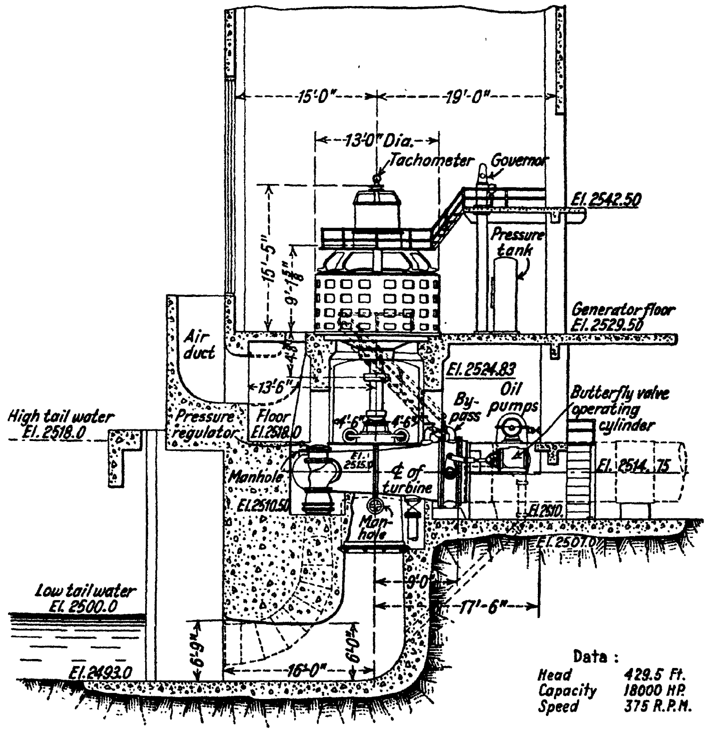


FIG. 383.—Vertical Cast-steel Spiral-cased Turbine for High Head, 18,000 h.p., 429-ft. ment with switchboard floor at exciter level, main floor at generator base, and turbine zontally with hydraulically operated butterfly valve, cast-steel casing made in three sec- removable cast-iron section below runner. Governor stands and controls located on

on the extended end of the generator shaft, the two generator bearings carrying the weight of the rotor and the impulse wheel. On this unit the regulating cylinder of the governor is connected directly to the needle, a governor-operated pressure regulator being provided so that when the needle is closed rapidly the pressure regulator opens simultaneously, preventing a serious rise in pressure. The pressure regulator then is caused to close slowly, so that the water will not continue to be wasted. Some reliable type of valve is usually required in an impulse turbine plant, as the penstocks are usually long and the pressure high. The majority of the plants use a special type of gate valve, although there are some installations where the needle type of valve is used.

Figure 389 shows an impulse wheel of 35,000-h.p. capacity under a head of 2100 ft., this being of the double overhung type, one 17,500-h.p. wheel being mounted on each end of the extended generator shaft. Each wheel end is provided with its separate and complete governor so that either end may be operated alone for light loads.



Head, 375 r.p.m. Kehin Denryoku Electric Power Co., Japan, showing three-floor arrange-
floor at cover plate level with an inspection floor below turbine. Penstock enters hori-
tions, pressure regulator with plate-steel discharge pipe, concrete-elbow draft tube with
switchboard floor, motor and impulse wheel-driven oil pumps located on turbine floor.

Figure 390 shows an arrangement of a vertical-shaft impulse turbine where two jets are used on the one disk, a pressure regulator being provided to prevent serious pressure rises. The generator is arranged very similarly to that on a cast-casing power house, the water being brought into the wheel through a penstock located below the floor, the revolving parts of the impulse wheel being covered in a waterproof chamber.

The various illustrations which have been shown above as illustrating

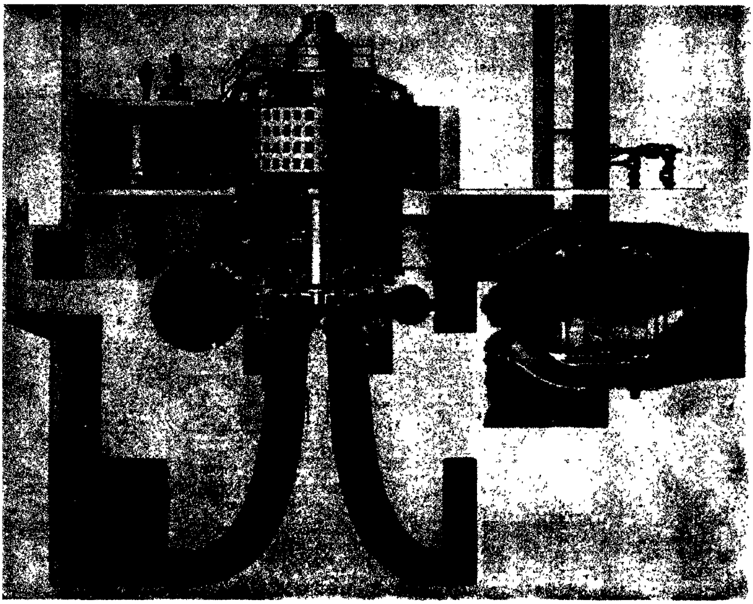


Fig. 384.—Section through 70,000 h.p. 213-ft. Head 107 r.p.m. Vertical Cast-steel Spiral-cased Turbine. Niagara Falls Power Company Units 19 and 20, showing concentric draft tube with high cone, labyrinth runner seals, adjustable lignum vitæ bearing and Johnson valve at casing inlet. (I. P. Morris).

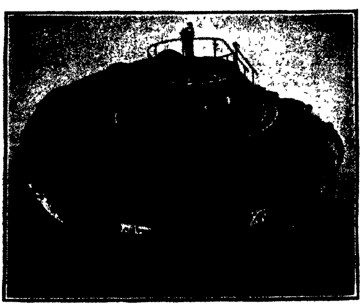


Fig. 385.—Cast-steel Casing for 70,000 h.p. Niagara Falls Power Company units 19 and 20, showing how sections of casing are fitted together, also arrangement of guide vane

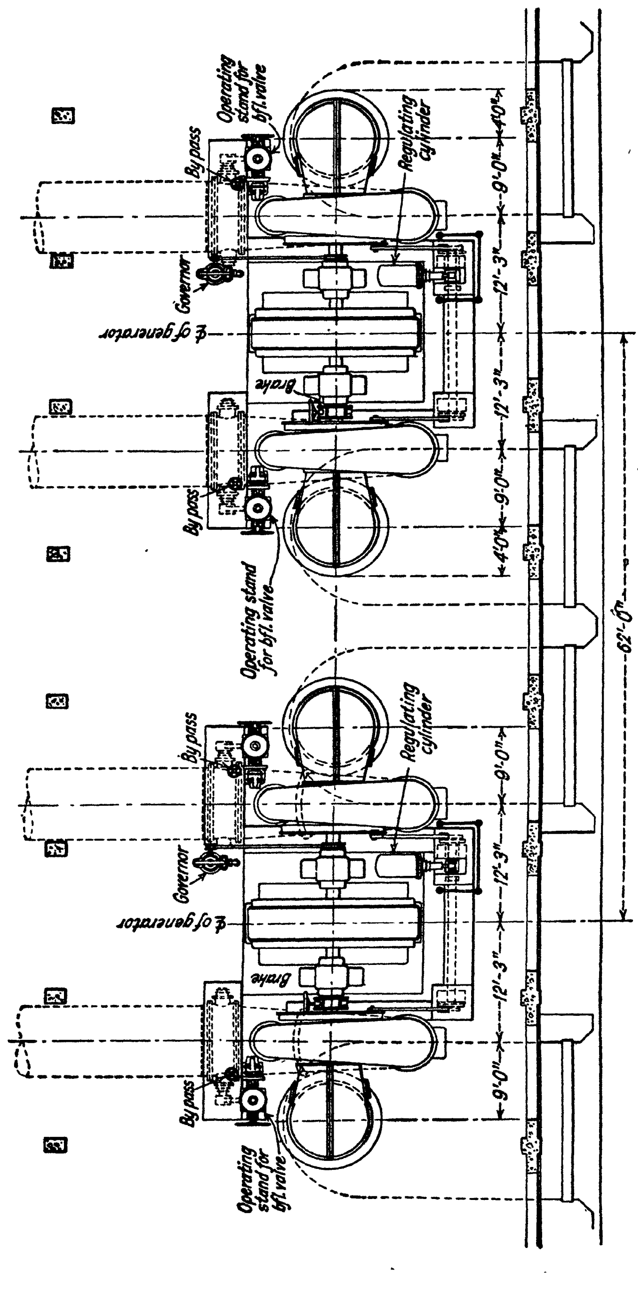


FIG. 386.—Plan View of Two 20,000 h.p. 200-ft. Head Double Overhung Cast-steel Spiral-cased Turbines, Showing Butterfly Valves, Governor and Regulating Shaft. Generator located between bearings with one runner overhung from each end of main shaft; all parts located on one floor, makes economical construction and is simple to operate. Baker River Development.

the five principal types of turbine do not by any means cover the subject, as there are many variations in the details and arrangement of the various parts.

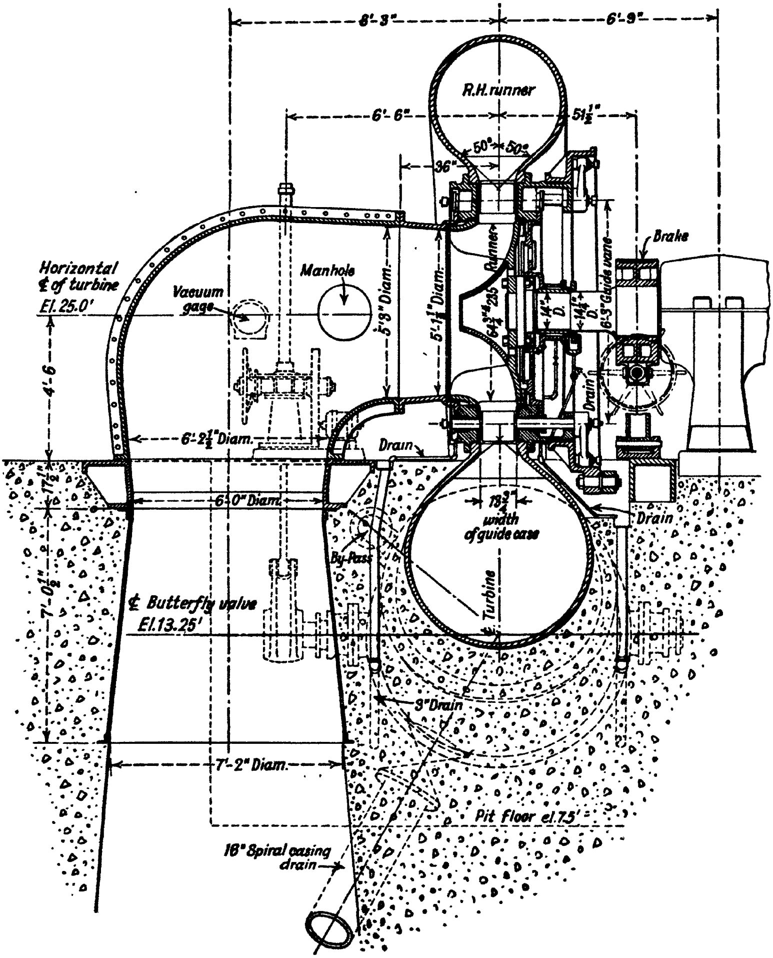


FIG. 387.—Section through 20,000-h.p. 200-ft. Head Double Overhung Cast-steel Cased Turbine, Showing Quarterturn Draft Tubes, Cast-steel Runners Bolted to Shaft, Flanges so that Runners May be Removed without Disturbing Shaft, Shifting Ring Located Outside of Guide Vane Stems and Friction Brake to Stop Unit. Baker River Development. (Allis-Chalmers Manufacturing Company.)

These illustrations, however, are intended as an outline or guide, the various arrangements and location of the detailed parts being discussed more fully in connection with the description of these detailed parts. These arrange-

ments illustrate simple and efficient settings which may be considered as good examples of present-day hydro-electric practice.

318. Selection of Equipment.—The usual problem which confronts an engineer in the first stages of selecting the proper hydraulic machinery is to determine which type and which setting will make the most economical development. The authors believe that the general outline given above will give him a fair understanding of the principal use for which each type is intended. There are, of course, special features in connection with almost every hydro-electric installation which must be studied and considered before a final decision is made. These, however, usually require the advice of the various manufacturers and designers, who should always be consulted before a final decision is reached. When the engineer has satisfied himself as to the type of turbine which he considers best suited to his needs, the next problem is to determine the proper rating of power and speed for his conditions.

Usually the available head is quickly determined. In the case of open-flume turbines or spiral-cased units with short penstocks there is little loss; the difference between normal head-water and normal tail-water may therefore be taken as the net effective head.

In the case of units that are fed through long canals or long pipe lines, careful computations should be made of the size of flume or penstock required, with figures showing the head lost in friction for the different amounts of water used. This matter is covered in more detail in Chapter IX. The matter of length of penstock in connection with the speed and voltage regulation of the plant must also be considered before a final decision is reached. This is covered in detail in Sec. 334.

In selecting the speed of a hydro-electric unit the cost and efficiency of the generator must be taken into consideration. For the average size generator the cost decreases as the speed is increased, although somewhere a point is reached where this ceases to be true, but this is far above the capacities and speeds met in average practice.

Another and far more important matter to be considered when determining the speed of a hydro-electric unit is the characteristics of the runner which will be used in the hydraulic turbine. This characteristic is defined mathematically by the formula:

$$N_s = \frac{RP^{1/2}}{H^{5/4}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (159)$$

where N_s = specific speed;
R = revolutions per minute, r.p.m.;
P = horse power per runner;
H = head in feet at which unit is operating.

This is termed "specific speed," and may be more clearly defined as the r.p.m. at which a runner would run if it were so reduced in proportion that it would develop one horse power under one foot head. Generally speaking, runners having a low specific speed are suited for high heads, and runners having a high specific speed are suited for low heads. High-

specific-speed runners of the Francis type, such as are used for low heads, have a discharge diameter greater than the inlet diameter; runners of about 42 specific speed, suitable for about 200-ft. head, have about the same inlet and discharge diameters; while lower-specific-speed runners for heads above 200 ft. have a smaller discharge diameter than inlet diameter. In high-specific-speed runners, the relative velocity between the water and the runner is greater than in low-specific-speed runners, and the velocity of the water discharged from the high-specific-speed runner is relatively greater than from the low-specific-speed runner. This is the principal reason why a high-specific-speed runner cannot be used for a high head; the velocity of the discharged water would be greater than the spouting velocity at 34-ft. head, so that if a draft tube were used and this velocity were regained, the resulting vacuum would approach 34 ft., although it could not exceed this vacuum, and as a result the column of water would break and serious pitting would occur. Also, high-specific-speed runners cannot be made strong enough for high heads. Similarly, low-specific-speed runners, while they will operate successfully at low heads, would result in such low speeds that generators would be excessively expensive.

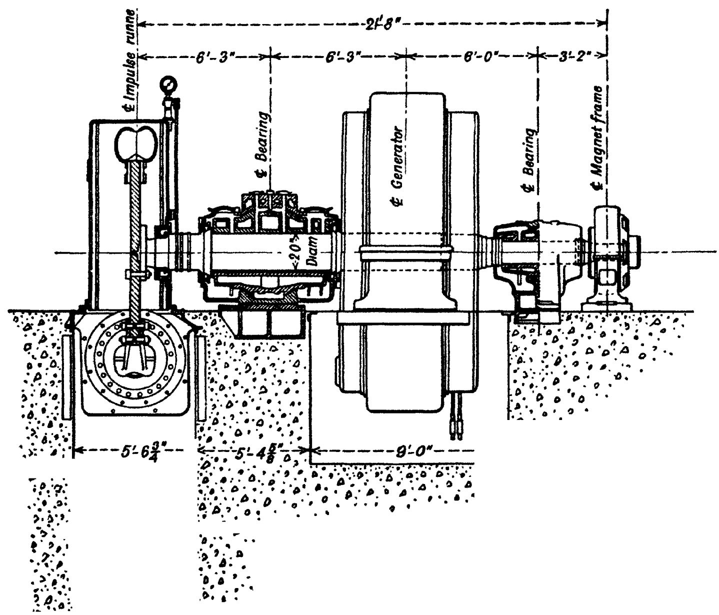
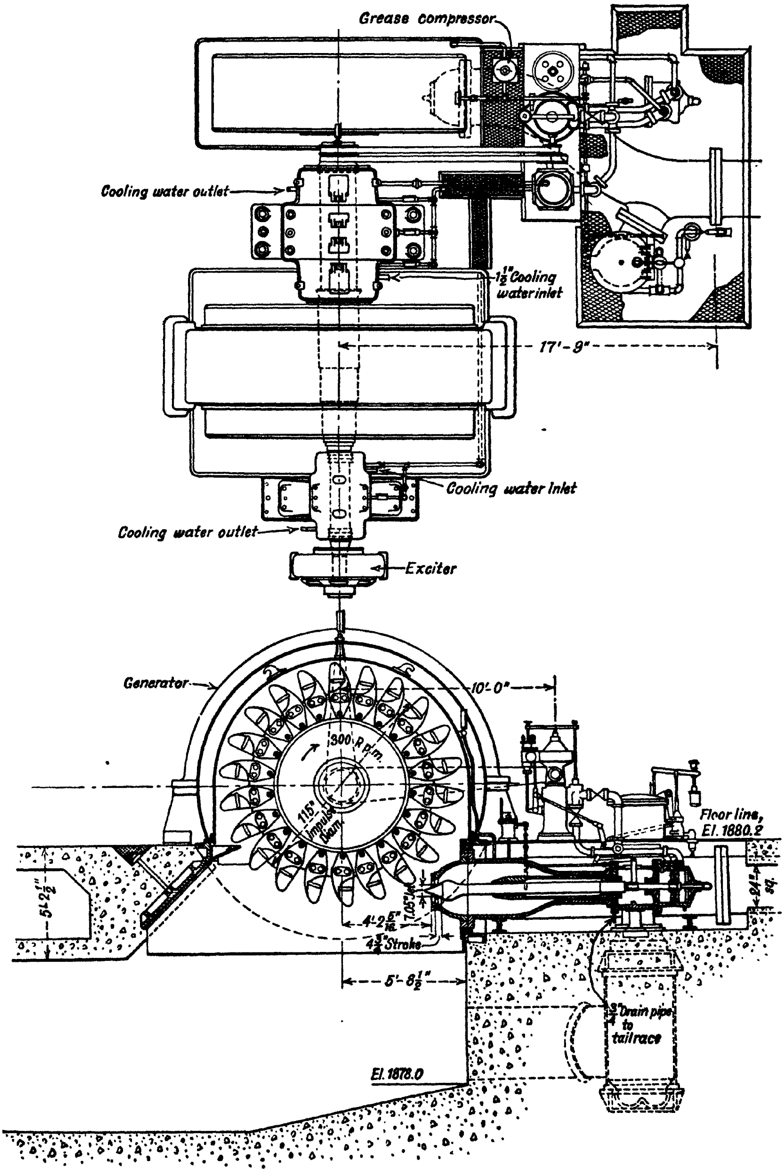


FIG. 388.—Single Overhung Impulse Turbine, 14,000 h.p., 1750-ft. Head, 300 r.p.m., Governor-controlled Pressure Regulator Discharging into Energy Absorber, Forged Generator Rotor Mounted between Bearings, Impulse Disk Overhung on End of Gas and Electric Company.



with Governor Operated Needle, Regulating Cylinder Mounted Directly on Nozzle Pipe, Steel Disk Bolted to Flange on Main Shaft, Cast Steel Buckets with Three Bolts Each, Shaft, Frequently Built with an Impulse Disk Overhung at each end. Western States (Allis-Chalmers Mfg. Co.)

A general formula from which the maximum safe specific speed for any given head may be computed is:

$$\frac{5050}{H + 32} \quad 19, \qquad (160)$$

where N_s' = maximum safe specific speed;
H = head in feet at which unit will operate.

Figure 391 shows this formula plotted in a full line for various heads. This formula has no theoretical derivation, but was developed by Forrest Nagler by plotting the characteristics of a large number of plants. He used specific speed as the vertical scale and effective head in feet as the horizontal

FIG. 389.—Power House View of Double Overhung Impulse Turbines, 35,000 h.p., 1900-ft. Head, 300 r.p.m. Southern California Edison Company, showing arrangement of wheels and generator with separate governor for each wheel side. Cast-iron lower and plate steel upper housing (Pelton Waterwheel Company).

scale, and drew a line separating those plants which on the whole operated satisfactorily from those plants which did not operate satisfactorily, the principal criterion of satisfactory operation being whether or not there was serious pitting of the runners. (See Report of National Electric Light Association for 1924, also Moody and Rogers paper, A. S. M. E., Vol. 47, No. 4, page 308.)

Pitting of hydraulic-turbine runners is an action which takes place during operation and by which the metal forming the buckets or blades of the runner disappears. It may be chemical or electrochemical action, but experience seems to indicate that where there are relatively high velocities and high vacuum, pitting will occur more rapidly. In cases where pitting is bad, runner blades having a thickness of over 1 in. are eaten through in less than one year's operation. Pitting occurs on runners of all common materials, bronze, cast steel, cast iron, and plate steel, no material having yet been found from which a satisfactory runner casting can be made which will absolutely resist this action.

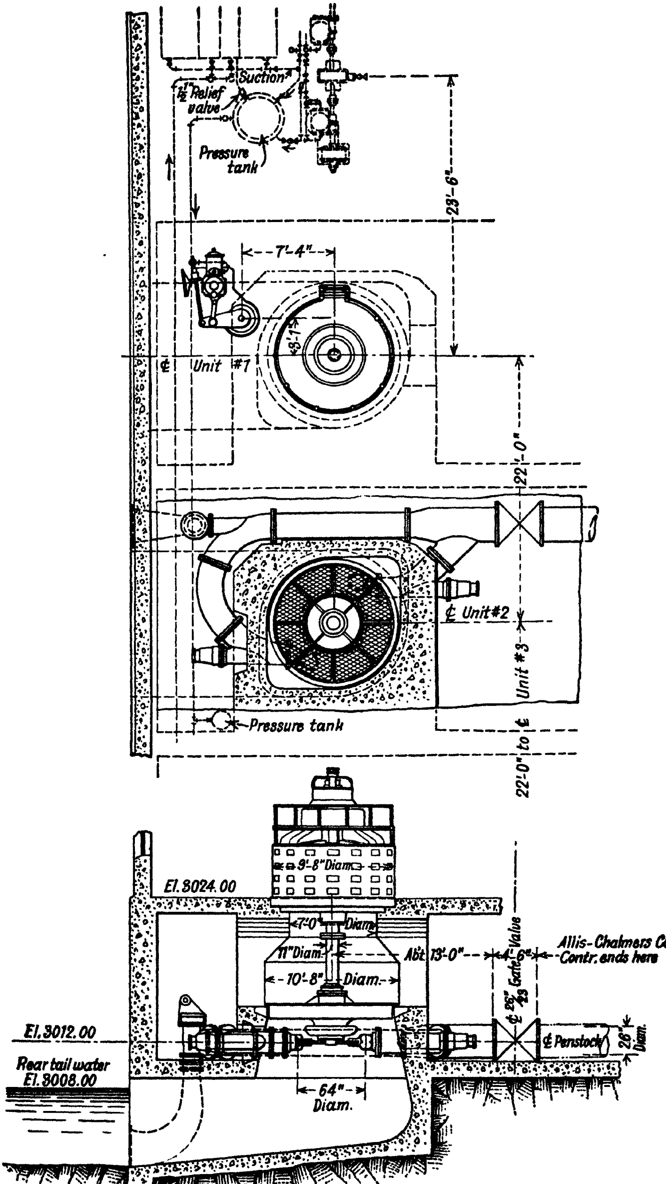


FIG. 390.—Vertical Shaft Two-jet Impulse Wheel, both Needles Operated from One Governor, Governor Controlled Pressure Regulator, Two-floor Station. Type of setting which is expected to become popular for large-capacity impulse wheels.

Pitted runners can be repaired by welding in additional metal when the runner material is of cast steel or cast iron. Bronze runners, if pitted, may also be built up, although it is quite difficult. Sometimes it is possible to stop local pitting by drilling a small hole through the runner blade at the pitted section. It is possible to select a specific speed which will not cause pitting, for any set of conditions for which a runner may be designed.

As more experience is gained and more efficient runners are developed, it is being found out that the specific speed which may be used without serious pitting can be increased somewhat above this curve for heads below 40 ft.

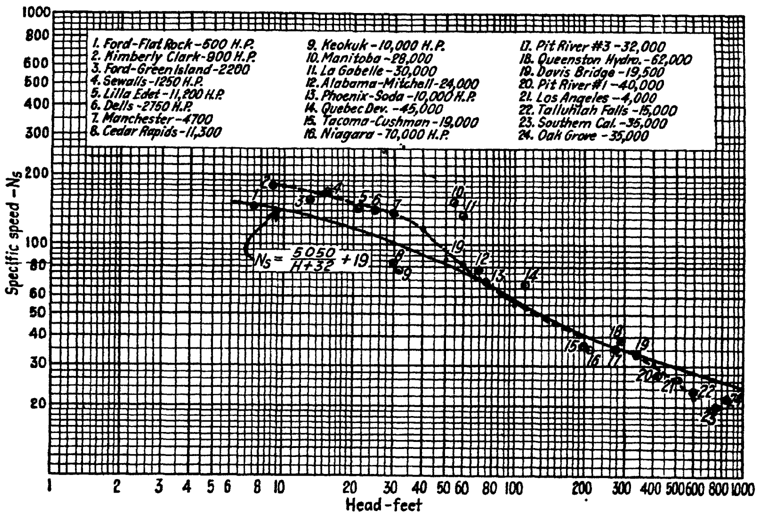


FIG. 391.—Full Line Shows Experience Curve for Specific Speed, Used Extensively for Determining Maximum Safe Specific Speed. Plants listed show recent practice. Dotted curve shows maximum safe specific speeds recommended at present time.

Figure 391 has plotted upon it the specific speed of a large number of the prominent plants which have been recently constructed and are now in operation. It will be observed that many of these plants have a specific speed considerably above that given by the experience curve. Not all of these plants operate satisfactorily. Some of them are pitting seriously.

The dotted line in Fig. 391 shows specific speeds that have been found to give satisfactory results at the given heads, provided the runner is of proper design and the draft head not over 12 ft. below the center line of the runner.

In each case, before definitely selecting a specific speed above this curve, especially for a head above 30 ft., the matter of reliability should be considered carefully, the increase in cost of a lower-speed generator being compared with the probable cost of frequent shut-downs for renewing or repairing the runner in case pitting does occur. An excellent rule to follow in determining what specific speed is safe for a given head is to ask: "Is this specific speed beyond the limit of good practice, and is there a plant of comparable characteristics

which has been in operation at this specific speed for a sufficient length of time to determine whether or not the runner will stand up without serious pitting?"

Experience has proved that the matter of specific speed and head does not entirely control pitting. The height above tail-water, or the vacuum on the runner, seems to influence pitting greatly, as is shown by the N. E. L. A. Report of 1924.

A good rule to follow is to keep the center line of the turbine not over 10 or 12 ft. above normal tail-water level, and under extreme conditions of low tail-water this should not exceed 16 ft. This applies to both vertical and horizontal-shaft units. If relatively high-specific-speed runners are used for an installation the draft head should be decreased to compensate for this.

For conditions where there is a large fluctuation in tail-water level due to floods or other causes, it is frequently impracticable to use horizontal-shaft units, as the tail-water might rise above the generator floor. The best solution for such conditions is to use a vertical unit with a long shaft and place the generator floor above high tail-water level. This may leave the turbine submerged during high tail-water periods; but the turbine will operate satisfactorily when submerged, although it may be necessary to provide gates for shutting off the draft tube and pumps for removing the water from the turbine if inspection or repairs are required during high tail-water periods.

The design of the runner may also have a great effect on pitting, as there are cases where runners having a specific speed considerably below that given on the experience chart have pitted badly. This would seem to be caused by improper design. This is more common on the high-head Francis turbines, that is, for heads above 200 or 300 ft., and it is for this reason that the majority of the high-head Francis runners are designed with a specific speed well below that indicated by the experience curve.

For heads below 30 or 40 ft. the experience curve may practically be disregarded when propeller runners are used, as the propeller-type runner has demonstrated that it can operate satisfactorily without pitting at specific speeds far beyond that indicated by the experience curve. It has been proved that propeller-type runners having the specific speed shown by the dotted line on Fig. 391 will operate satisfactorily at these heads without pitting, provided that the distance from the runner to the tail-water is not too great. This distance should be determined by the manufacturer, and it should decrease as the head increases. The vacuum on the runner should not exceed 24 ft. of water, this including both static and velocity head.

Having determined the proper specific speed, and knowing the head and the horse power to be developed per unit, the engineer will find Fig. 392 of assistance in rapidly selecting the proper speed for various conditions. The scale along the top of the sheet shows the specific speed, that along the bottom shows revolutions per minute. Having determined the proper specific speed, find the intersection of head and specific speed and then follow over horizontally either to the right or left to the intersection with the horse-power curve, then drop vertically. The revolutions read will be the speed corresponding most nearly to the specific speed at the head and horse power given. Similarly, if any three of the variables are known the fourth can be readily

determined. This curve greatly facilitates the solution of problems of this nature as it eliminates the necessity of computing the horse power and revolutions at 1-ft. head. The problem illustrated in light lines in Fig. 392 shows that for 70 ft. head and a specific speed of 70, a 20,000-h.p. unit will run at 100 r.p.m. The dotted line on this curve is the specific-speed formula given above which is the maximum safe limit of specific speed that may be selected without first conferring with reliable water-wheel builders.

Another point to be considered in selecting the specific speed for any set of conditions is the matter of part-load efficiency. Best efficiency at the normal specific speeds used has been developed to such a point that, for runners having a specific speed between 20 and 160, a maximum efficiency of 90 per cent or over may be expected, for good conditions; and up to 200 specific speed, efficiencies of over 85 per cent may be expected. In other words, for a considerable change in specific speed the best efficiency is not materially different. However, the part-load efficiency differs greatly for different specific speeds and the percentage of load at which the best efficiency occurs also varies.

The table below shows approximately the difference in the efficiency of three different runners, $N_s = 4$ being an impulse wheel, $N_s = 50$ and $N_s = 100$ being Francis runners, and $N_s = 150$ being a propeller runner.

EFFICIENCY

Load	$N_s = 4$	$N_s = 50$	$N_s = 100$	$N_s = 150$
Full	86.0%	86.0%	85.5%	84.0%
95%	87.0	90.5	91.0	90.0
90	87.5	92.0	92.5	89.5
85	88.0	92.5	91.5	88.0
75	89.0	91.5	89.0	82.5
50	87.0	87.5	78.0	66.0
25	82.0	70.0	60.0	48.0

Where there are a large number of units in the plant, or where pondage is ample and the unit is connected to a large system, the matter of part-load efficiency is not so vital, as here it is possible to operate the units carrying the load, at their point of best efficiency. The conditions are entirely different, however, in a station having but one unit. If this one unit must be operated to deliver whatever power is required, or to utilize the flow of the stream, it will mean that at certain times of the year and certain times of the day this unit will be operating at half load or lower. Here the part-load efficiency is of vital importance.

The fact that low-specific-speed runners have their point of best efficiency at from 80 to 85 per cent of full load may make it necessary, in high-head plants, to install units considerably larger than actually required in order that they may be operated the greater part of the time at their point of best efficiency.

The trend of modern design has been toward the inter-connection of

systems and units. This greatly simplifies conditions by allowing each unit to be operated at its point of maximum efficiency, so as to produce the greatest number of kilowatts from the water available. (See F. S. Allner's paper, A. S. M. E., Vol. 47, No. 9, page 727.)

There is frequently cause for considering the part-load efficiency of low-

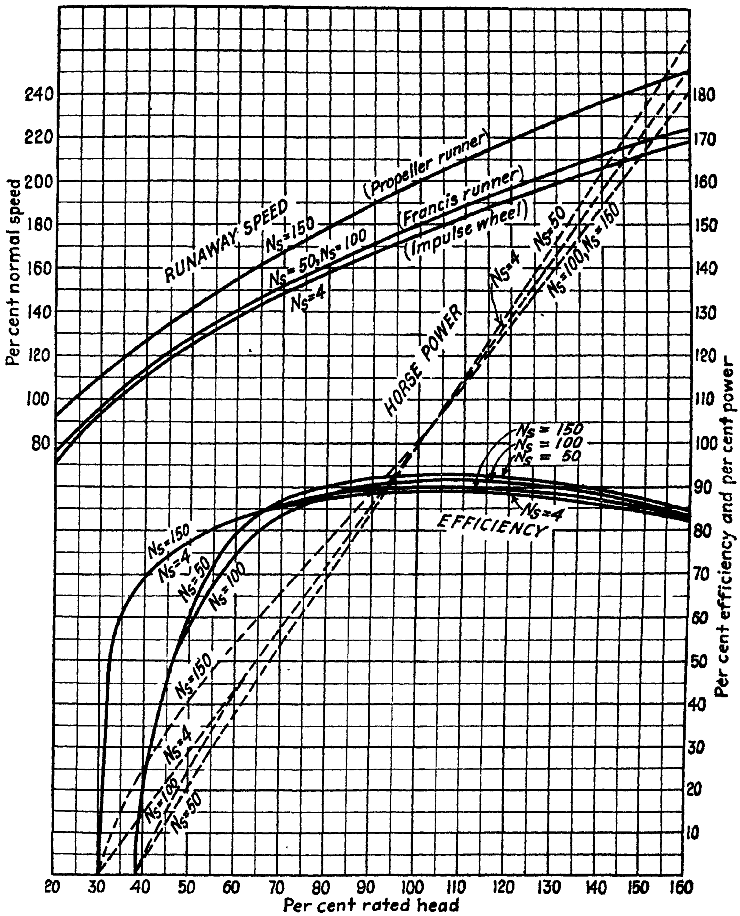


FIG. 393.—Efficiency and Horse Power of Various Specific Speed Runners Operating at Constant Speed under Abnormal Heads. Also, runaway speeds under abnormal heads.

head units, in deciding whether a Francis-type runner of 90 to 100 specific speed or a propeller-type runner with a specific speed of 150 should be used. Where the plant is an isolated one and efficiency is vital, it sometimes works out that the slower-speed, although more expensive, Francis-type unit is the

best investment. This is the exception rather than the rule, however, as usually the saving to be made by using the higher-speed unit, especially the higher-speed generator, more than outbalances the gain in efficiency at part load.

The efficiencies tabulated above are approximately those that may be expected in installations which are made under favorable conditions, that is, where there is ample room, so that the intake velocities are reasonable, and where there is sufficient depth and width of tail race, so that a proper regain may be accomplished in the draft tube. The matter of setting is very important, especially for higher-specific-speed runners. The efficiencies shown on this table cannot be expected unless the station is properly designed.

Figure 393 shows how the efficiency of runners of different specific speeds varies under varying conditions of head at constant speed. The runaway speed and the percentage of normal horse power are also shown.

Special attention is called to the fact that the propeller-type runner of 150 specific speed holds up its horse power to a greater extent under decreased head conditions. At 40 per cent of normal head, the propeller-type runner will still be delivering about 25 per cent of its rated capacity; whereas the Francis-type runner will scarcely be able to produce any useful power, and, as the head is further decreased, it will not be possible to bring the Francis-type runner up to synchronous or normal speed. On this same sheet the efficiencies of the various types of runners are plotted. It will be noted that the efficiency of the propeller-type runner holds up to a greater extent under reduced heads of 40 to 70 per cent of normal.

Some of the principal differences between the Francis-type and the propeller-type units have been covered in the preceding paragraphs. Summing these up briefly, the propeller unit is suitable for medium and low heads. It enables us to obtain higher speed, and lower-priced and more efficient generators. The efficiency of the propeller turbine is very nearly equal to that of the Francis-type runner at its point of best efficiency, although at fractional loads the propeller runner is less efficient. On account of the relatively high velocities in the discharge from the propeller-type runner, the design of the draft tube or regaining device is more important than on the Francis-type unit. By improper setting, it is easily possible to reduce the capacity of a propeller runner 15 per cent with a loss in efficiency of 8 to 10 per cent. In order to obtain the best results and the most efficient setting, the water-wheel builders should be consulted; and in special cases a model of the complete setting and draft tube should be designed in order to insure that the losses will not be greater than anticipated. Recently there has been an increase in the tendency to test models of plants and setting; this practice has resulted in a very great increase in efficiency and in a knowledge of what conditions are most beneficial in designing settings.

When the engineer has satisfied himself that he has made the proper selection of power and speed for the conditions of head flow and load which confront him, the next matter of interest to him is the size of the machine. Figure 394 shows the approximate discharge diameter of the runners required to develop various horse powers under different conditions of head. Discharge

diameter in inches forms the vertical scale, and feet effective head forms the horizontal scale, the various arrows representing different capacities of turbines, the discharge diameter of which it is desired to determine. The discharge diameter is measured at the bottom of the runner, that is, at the point where the water discharges from the runner.

While this curve does not indicate the specific speeds for the various heads, the runner diameters have been so selected that they correspond to the average type of runner which is used for the particular conditions of head. While the discharge diameter is not always the principal dimension used in referring

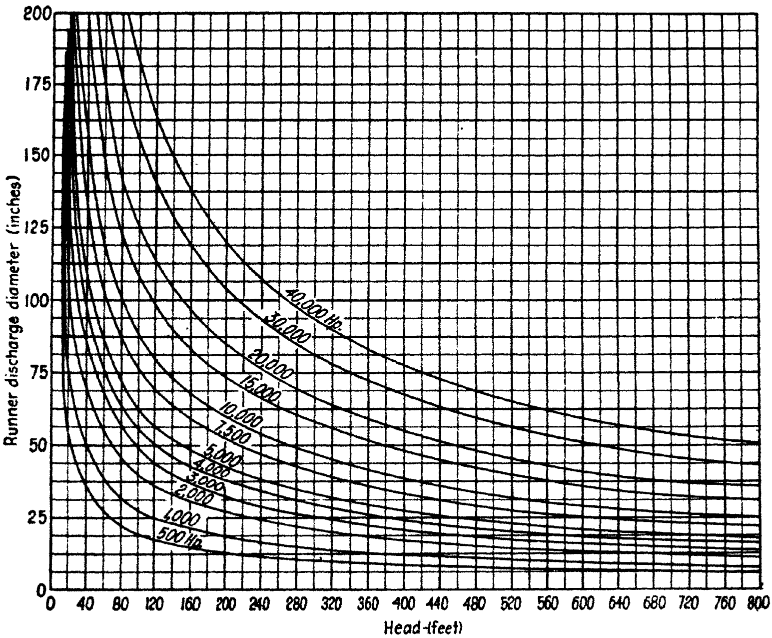


FIG. 394.—Discharge Diameters of Runners Required to Develop Various Capacities under Different Heads. Based on normal specific speeds.

to hydraulic-turbine runners, it is really the vital one since it governs the size of the machine, and the width of the tailrace. Figure 395 gives the dimensions, as well as the characteristics, of runners of various specific speeds based on a discharge diameter of 100 in. These data are not intended to enable one to design a hydraulic turbine. The dimensions are approximate and are of assistance only in enabling an engineer to lay out a power station in a preliminary manner.

Assuming that we know the head and horse power of the proposed plant, the unit horse power, or horse power at 1 ft. head, may be obtained by dividing the horse power by the three-halves power of the head. Assuming the

head to be 180 ft., and the horse power 35,000, the unit horse power (h.p.$_1$) is 14.5. From Fig. 391 we find that the maximum safe specific speed for 180 ft. head is 42, so we may assume 40 N_s for preliminary studies. Figure 395 shows that the h.p.$_1$ of a 100-in. discharge-diameter runner of 40 N_s is 10, so that

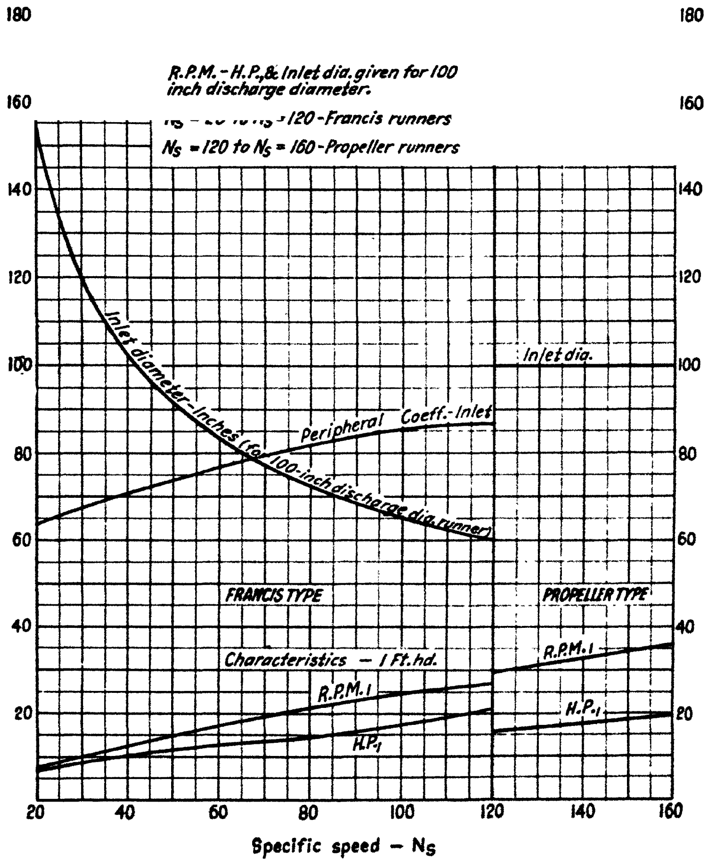


FIG. 395.—Curves Showing Characteristics of Hydraulic Turbine Runners of Various Specific Speeds. R.p.m., h.p. and inlet diameter given for 100-in. discharge diameter.
N_s = 20 to N_s = 120-Francis runners.
N_s = 120 to N_s = 160-propeller runners.

to develop 14.5 h.p.$_1$, we shall require a runner of 120-in. discharge diameter, since the horse power varies as the square of the runner diameter. This may be checked by Fig. 394, which shows that to develop 35,000 h.p. under 180-ft. head requires a 120-in. discharge-diameter runner.

Figure 395 shows that the inlet diameter, which is the diameter at the center line of the guide vanes or distributor, of a 40 N_s runner is 102½ per cent of the discharge diameter, or 123-in. inlet diameter for a 120-in. discharge-diameter runner. The r.p.m. at 1 ft. head of the 100-in. discharge-diameter runner of 40 N_s is given in Fig. 395 as 12.6; and since the r.p.m. varies inversely as the runner diameter, the r.p.m.$_1$ of the 120-in. runner will be 10.5; multiplying by the square root of 180-ft. head, we find the proper speed will be 141 r.p.m. Since 141 r.p.m. is not a synchronous speed for 60-cycle generators, we must drop to the next lower synchronous speed, which is 138½ r.p.m.

"Peripheral coefficient" is the term applied to the relation between velocity of runner inlet and spouting velocity of water. The spouting velocity of water may be computed from the formula:

$$V = \sqrt{2gH}, \qquad (161)$$

where V = spouting velocity, feet per second;
g = acceleration due to gravity, 32.2 ft.-sec. per sec.;
H = head in feet.

For 180-ft. head, the spouting velocity is 107.5 ft. per second. The 123-in. inlet diameter runner, revolving 138½ times per minute, is traveling 74.4 ft. per second or 69.2 per cent of the spouting velocity of the water, which checks closely the peripheral coefficient as obtained from Fig. 395 for a specific speed slightly below 40.

The characteristics of propeller runners are given on the right-hand side of Fig. 395, and may be computed in the same manner as the problem illustrated above. The characteristics of impulse wheels are not so readily illustrated in curve form, but a sample solution is given in Sec. 332.

Since the runner diameter and type practically determine the entire turbine setting, by means of the above-described method any engineer may compute closely the size and type of runner best adapted to his conditions. In all cases, however, the selection of speed and type should be discussed with the turbine builders, as some special conditions or special development may make another type or speed more economical.

319. Homologous Equations.—Knowing the efficiency, speed, power, and discharge of a runner of a given diameter at a given head, we may calculate directly from the following equations, the speed, power, and discharge of a homologous runner of a different diameter, under a different head for the same efficiency.

For constant diameter,

$$\frac{P_2}{P_1} = \left(\frac{H_2}{H_1}\right)^{3/2}, \qquad (162)$$

$$\frac{R_2}{R_1} = \left(\frac{H_2}{H_1}\right)^{1/2}, \qquad (163)$$

$$\frac{Q_2}{Q_1} = \left(\frac{H_2}{H_1}\right)^{1/2}, \qquad (164)$$

For constant head,

$$\frac{P_2}{P_1} = \left(\frac{d_2}{d_1}\right)^2, \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (165)$$

$$\frac{R_2}{R_1} = \frac{d_1}{d_2}, \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (167)$$

$$\frac{Q_2}{Q_1} = \left(\frac{d_2}{d_1}\right)^2. \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (167)$$

where P_1 and P_2 = horse power for different conditions;
d_1 and d_2 = runner diameter for different conditions, in inches;
H_1 and H_2 = Head in feet, for different conditions;
Q_1 and Q_2 = discharge in C. F. S., for different conditions;
R_1 and R_2 = revolutions per minute.

These six equations may be summarized as follows:

The horse power of a runner changes as the square of its diameter, as the three-halves power of the head, or directly as the discharge.

The revolutions per minute of a wheel should be changed in proportion to the square root of the head and inversely in proportion to the diameter of the runners.

The discharge from a wheel will vary in direct proportion to the power, as the square of the runner diameters, and as the square root of the head.

For example, let a known runner have the following characteristics:

$$\begin{aligned} P_1 &= 1000 \text{ h.p.}; \\ d_1 &= 50 \text{ in.}; \\ H_1 &= 100 \text{ ft.}; \\ R_1 &= 180 \text{ r.p.m.}; \\ Q_1 &= 100 \text{ cu. ft. per sec.} \end{aligned}$$

Let it be desired to know the required diameter, speed, and discharge of a runner to operate at 150-ft. head and give 2000 h.p. at the same efficiency.

From Eqs. (162) to (164), the characteristics of the known runner under 150-ft. head would be

$$P_2 = 1000 \left(\frac{150}{100}\right)^{3/2} = 1837 \text{ h.p.};$$

$$R_2 = 180 \left(\frac{150}{100}\right)^{1/2} = 220 \text{ r.p.m.};$$

$$Q_2 = 100 \left(\frac{150}{100}\right)^{1/2} = 122.5 \text{ cu. ft. per sec.}$$

To give 2000 h.p. under this head of 150 ft., the turbine must have the following characteristics, from Eqs. (165) to (167):

$$d_2 = 50\left(\frac{2000}{1837}\right)^{\frac{1}{2}} = 52.2 \text{ in.}$$

$$R_2 = 220\left(\frac{50.0}{52.2}\right) = 210 \text{ r.p.m.}$$

$$Q_2 = 122.5\left(\frac{52.2}{50.0}\right)^2 = 133.4 \text{ cu. ft. per sec.}$$

320. The Design of the Water Passages.—The matter of water velocity at the inlet of the turbine is of great importance and has an appreciable effect

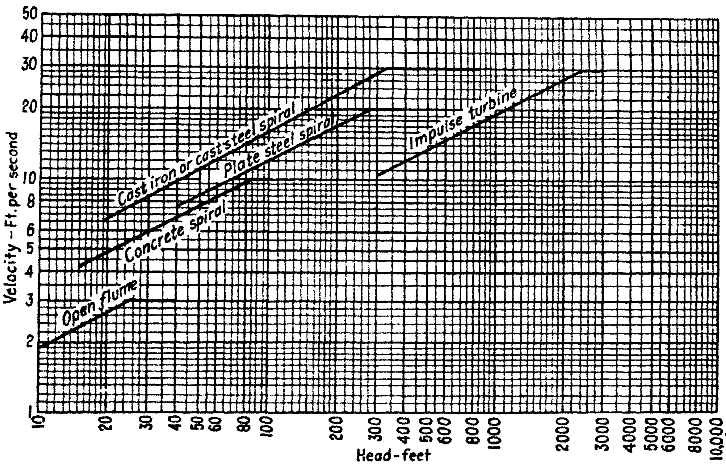


FIG. 396.—Curves Showing Allowable Water Velocities in Various Types of Casings for Reaction Wheels and Inlet Pipes of Impulse Wheels.

upon the full-load and part-load efficiency of the unit. Fig. 396 shows safe velocities in casings and flumes for the various types of units. For open-flume turbines, when the water is not guided, the inlet velocity should not exceed 7½ per cent of the spouting velocity of the water; and for heads above 20 to 25 ft. a maximum velocity of 3 ft. per second should not be exceeded, because in open flumes of this type there is a tendency for whirlpools and eddies to form in water traveling at high velocity. These enter the runner and cause disturbances, or may set up vibrations which may be felt on the unit or even in other parts of the building.

For units equipped with concrete spiral casings, the velocity in the casing should not exceed 13 or 14 per cent of the spouting velocity in the water, or a maximum of 10 ft. per second. This value of 10 ft. per second is determined more for the safety of the concrete than for the matter of runner efficiency,

and velocities of 5 or 6 ft. maximum are recommended wherever possible. A carefully made concrete spiral casing may duplicate the form of a cast casing, but there may be some danger of wearing away the concrete if the velocity exceeds 10 ft. per second.

Turbines equipped with plate-steel spiral casings may use up to 15 per cent of the spouting velocity of the water in the casing, a maximum of 20 ft. per second being the limit for good practice at the present time.

Casings of cast iron or cast steel use up to 20 per cent of the spouting velocity, a maximum velocity of 30 ft. per second being allowable under heads of 400 ft. or more. The question of velocity in the spiral casing is largely a matter of design, but it has been found that when the velocities in the casing are excessive the full-load efficiency of the unit may be seriously affected. It is difficult to determine what the effect of casing velocity will be without making field tests, as tests on models even though proportioned exactly to size, are not absolutely reliable in this respect, unless they are run under the head conditions existing in the plant, which is not usually the case. Similarly, the matter of hydraulic radius seems to have a considerable effect. The velocities shown in Fig. 296, however, may be used with safety; and, if the water passages are so designed that the full-load velocity does not exceed those given here, satisfactory results will be obtained.

The velocity in the nozzle pipe of an impulse turbine is usually kept below 10 per cent $\sqrt{2gh}$, and a better value is $7\frac{1}{2}$ per cent, with not over 30 ft. per second maximum. Higher velocities may cause distortion of the jet, especially if there are bends in the nozzle pipe.

In considering the matter of water velocity, especially in open-flume installation, the depth of water over the top of the wheel case should receive attention. With slight submergence, there is a tendency for air to be sucked into the wheel, which may seriously impair the capacity of the unit and may sometimes cause disturbance in the power output. Under lower-head conditions, say for below 15 ft., difficulty is frequently experienced in securing sufficient submergence. There should be a depth of water over the wheel of at least half the runner diameter. However, this is not found sufficient in some cases where flume velocities are high or disturbed conditions exist, and it has been found necessary to use wooden floats to prevent the turbine from drawing air. With the concrete spiral-case setting, the siphon construction may be used. This allows the wheel casing to be set very close to or even above head-water level, the water being drawn up by an ejector and air kept out by having a sealed chamber over the wheel.

The matter of spacing of rack bars at the intake of a turbine depends on local conditions and the type of foreign material which the water may bring down. Under no conditions should rack bars be spaced more than 5 or 6 in. apart, even with the largest runners, as timbers that go through these openings can do a great deal of damage. For propeller-runner plants, the rack spaces may be one-twentieth of the runner diameter, with a minimum space of 2 in. and a maximum space of 6 in. For reaction or Francis-type runners, a rack spacing of one-thirtieth of the discharge diameter of the runner, as determined from Fig. 394, may be used, with a minimum spacing of $1\frac{1}{4}$ in.,

and a maximum spacing of 4 in. For impulse wheels. a spacing of 1½ to 2½ in. is allowable.

In recent years there has been a great deal of activity in regard to the design of draft tubes, the trend of design leading toward the concentric type or which the hydraucone (Fig. 381) and spreading tube (Fig. 384) are excellent examples. For the higher-specific-speed runners, the matter of draft-tube efficiency is very important. When one considers that with a runner having a specific speed of 150, nearly 50 per cent of the energy is still in the water when it leaves the runner, the necessity for efficient regain can be realized. A long, straight, tapered draft tube is probably the most efficient regaining device, but the cost of excavation and the length which this type of tube would require for large-sized units make it almost prohibitive. The best substitute seems to be the concentric type. The width required for an

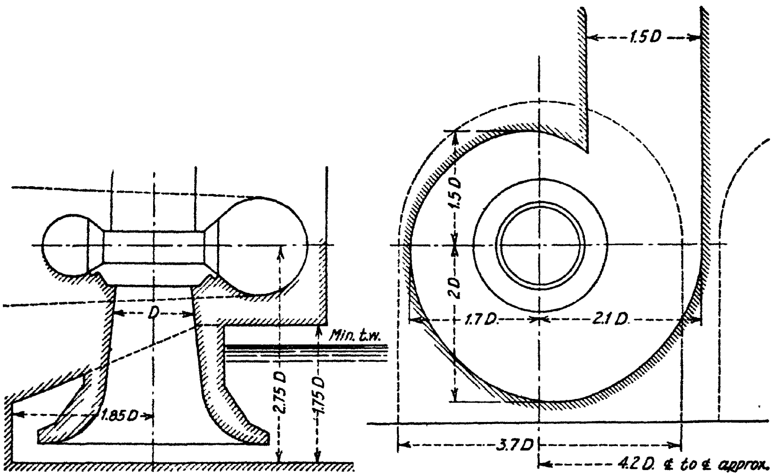


FIG. 397.—Approximate Power House Dimensions Based on Runner Discharge Diameter.

efficient type of concentric tube is about 3.7 times the discharge diameter of the runner, as shown on Fig. 397. The depth, from the bottom of the runner to the bottom of the tail race, should be at least twice the discharge diameter of the runner. For an average type of turbine, this will bring the bottom of the tail race about 2.75 diameters below the center of the guide case. Fig. 397 shows the approximate dimensions required in both tail race and casing in order to secure satisfactory water velocity, all these dimensions being based on the discharge diameter of the runner, which may be obtained approximately from Fig. 394 or computed by the method described in Sec. 318. These dimensions are not exact, but a draft tube designed with these distances will permit the installation of a very efficient regaining device, such as is required for high-speed or propeller runners and is preferable for practically all types of runners. For open-flume and concrete spiral-cased turbines, the draft-tube width will determine the spacing of units; but on plate-steel and

cast spiral-cased units, the casing width usually determines the spacing. The casing-inlet diameter must be checked for velocity, but the dimensions from Fig. 397 will give approximately the over-all space required.

321. Runaway Speed and Hydraulic Thrust.—In selecting the ratings and capacities of hydro-electric units, consideration must be given to the relation between the capacity of the generator and that of the hydraulic turbine.

The generator must be designed to stand the full runaway speed of the turbine to which it is to be connected, under the maximum head conditions at which the unit may operate. The ordinary type of reaction turbine will reach about 180 per cent of its normal speed at runaway; but if the maximum head on the turbine sometimes reaches 15 per cent above normal, then this same turbine will reach about 193 per cent of its normal speed at runaway and

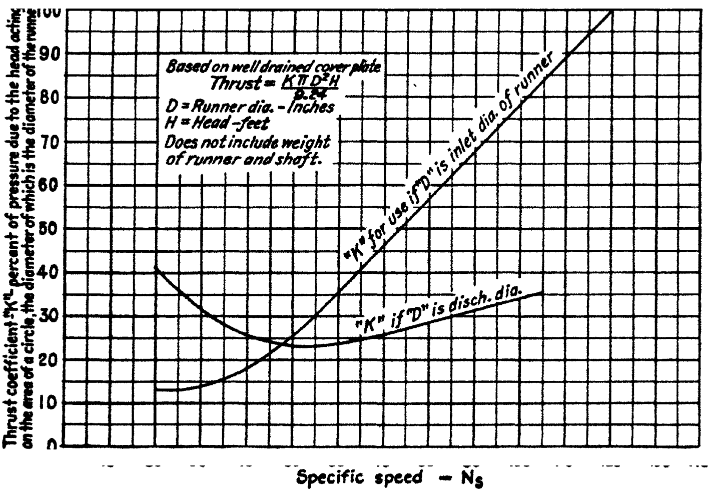


FIG. 398.—Hydraulic Thrust Chart.

the generators should be designed to stand these maximum conditions. Fig. 393 shows the percentage of runaway speed plotted against percentage normal head. The characteristics of Francis runners having a specific speed of 50 and of 100 are very similar in this respect; the propeller type of runner of 150 specific speed has a runaway about 20 per cent greater. On this same curve is shown the variation in horse power of various types of runners under varying head conditions.

The usual design of vertical units is such that the weight of the revolving parts of the hydraulic turbine, including the runner and shaft, as well as any additional downward thrust caused by the water pressure, is carreid by the thrust bearing, usually mounted on the upper bridge of the generator. The generator thrust bearing should, therefore, be designed with ample surplus capacity to take care of this additional load. Figure 398 gives the coefficients to be used in computing the hydraulic thrust, these coefficients being a per-

centage of the total pressure due to the head multiplied by the area of the runner. These curves are plotted for both the inlet diameter of the runner and the discharge diameter of the runner, and vary with the specific speed, specific speed being the horizontal scale. Propeller runners exert a thrust equal to the full area multiplied by the full head, so that the coefficient becomes 100 per cent when propeller runners are used. Since the inlet and discharge diameter of propeller runners are the same, the coefficient is 100 per cent in both cases.

$$T = \frac{\quad}{9.24} \qquad (168)$$

where T = hydraulic thrust, in pounds;
K = thrust coefficient (see Fig. 398);
D = runner diameter, in inches (see Fig. 394);
H = head in feet of water.

FIG. 399.—Cast-steel Runner, 176-in. Inlet Diameter, 37 Specific Speed for 70,000 hp., Niagara Falls Turbine. Runner bolted to forged flange on main shaft (Allis-Chalmers Mfg. Co.)

These values of hydraulic thrust are only applicable to turbine units so designed that the space between the crown of the runner and the cover plate is amply drained so that excess pressure will not be allowed to build up. The majority of turbines built to-day are so designed. The values computed by using these coefficients do not include the weight of runner and shaft, which should be obtained from the turbine manufacturer. Under ordinary condi

tions, the weight of runner and shaft will amount to 25 to 40 per cent of the hydraulic thrust.

322. Runners.—Probably the most important member of hydraulic turbine machinery is the runner. Figure 399 shows a cast-steel runner of 165-in. discharge diameter and about 37 specific speed. Note that the inlet diameter is slightly larger than the discharge. This runner develops about 83,000 h.p. under 213-ft. head at 107 r.p.m., and power-house tests show that its efficiency is over 93 per cent. This runner is installed in the unit shown in Fig. 381 and is bolted to the forged flange on the main shaft.

Figure 400 shows a cast-iron runner of about 58 specific speed, this being

FIG. 400.

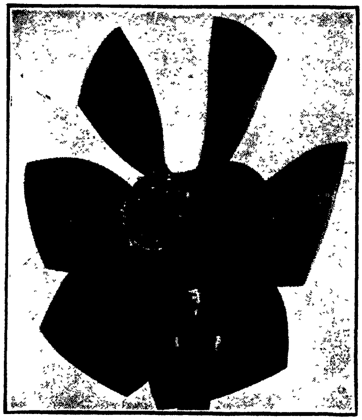


FIG. 401.

FIG. 400.—Cast-iron Runner of 60 Specific Speed for 30,000 hp. Muscle Shoals Turbine. Shows keyway and taper fit for connection to mainshaft. (I. P. Morris.)

FIG. 401.—Cast-steel Propeller Runner, 135 Specific Speed of 30,000 hp., 60-ft. Head LaGabrelle Turbine. Six blades cast in one piece, keyed to tapered mainshaft. (I. P. Morris, built by Dominion Engineering.)

for the 30,000-h.p., 95-ft. head, 100-r.p.m. units of the Muscle Shoals plant. The runner is pressed and keyed on to the tapered main shaft, the unit shown in Fig. 377 being of the concrete spiral-cased type, probably the highest-head concrete spiral-cased units ever built in large size.

Figure 401 shows a 189-in. cast-steel propeller runner of the Moody design, having a specific speed of 125 and developing 30,000 h.p. under 60-ft. head at 120 r.p.m. This is the largest-capacity, as well as the highest-head, propeller-type unit in the world. The runner has six blades cast integral and is keyed to a tapered main shaft, a section of the unit being similar to that shown in Fig. 376, but of heavier construction for the higher head. Fig. 402 shows a 124-in. cast-iron propeller runner of the Bell design, having a specific speed of 152. This is a three-blade runner, each of the blades covering approxi-

mately one-third of the area. The guide case, speed ring, pit liner, and draft-tube liner of the concrete spiral-cased turbine in which this runner is to be used are shown in the background of this picture. This unit is rated 3600 h.p., 22½-ft. head, at 120 r.p.m.

Figure 403 shows two 156-in. cast-steel propeller runners of the Nagler design, having a specific speed of 152, designed to develop 2200 h.p. at 13-ft. head and 80 r.p.m. These runners have four blades each, and in this case each blade, with one-quarter of the hub, was cast separately, the four were bolted together, and then the runner was bolted to a forged flange on the main shaft. This, the first large propeller-runner installation of the concrete spiral-

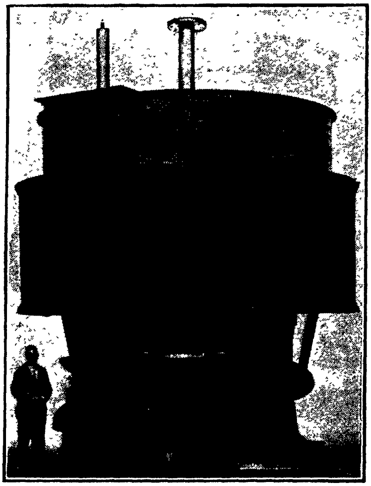


FIG. 402.

FIG. 403.

FIG. 402.—Cast-iron Propeller Runner and Complete Turbine, 140 Specific Speed for 3600 hp., 22½-ft. Head. Norman dam turbines. Three-bladed runner, keyed to main-shaft, cast-iron speed ring, plate-steel draft tube and pit liners. (S. Morgan Smith Co.)

FIG. 403.—Cast-steel Runners of 150 Specific Speed for 2200 hp. 13-ft. Head, Ford Motor, Green Island Turbines. Four blades cast separately, bolted together and to forged flange of main shaft. (Allis-Chalmers Mfg. Co.)

cased type was put in service in 1922, although smaller runners of this type have been in service since 1916.

Francis runners having a specific speed of 20 to 120 may be constructed of cast iron, cast steel, or bronze, or with cast-iron or cast-steel hubs and rims and plate-steel blades. For high-head conditions, either bronze or cast-steel runners are recommended, as the strength of cast iron is not sufficient for heads above 250 ft. For low-head conditions the plate-steel blades have been used successfully up to 75 ft. This construction greatly reduces the cost of building the runner and is entirely suitable for these conditions, as the plate steel is an excellent metal for this purpose, and by proper treatment a practically

perfect bond can be made between the end of the blades and the cast-iron or cast-steel hub.

Propeller-type runners for low heads are usually constructed of cast iron. For heads above 20 ft. they are usually made of cast steel as this is a much more reliable material.

323. Main Shaft.—The main shaft of the hydraulic turbine must transmit the full power of the turbine under the maximum head conditions at which the turbine will operate. In addition to the actual torque, it must carry the downward weight of the runner itself and the additional hydraulic thrust. If the main shaft is designed for a normal full-load stress under torsion alone of approximately 4000 lb. per square inch, there is usually a sufficient margin of strength to carry the additional thrust loads.

The approximate diameter of shaft required for a stress of 4000 lb. per square inch may be computed from the following formula:

$$d = 4\tfrac{1}{2}\left(\frac{P}{R}\right)^{1/3}, \qquad (169)$$

where d = shaft diameter in inches;
P = horse power;
R = revolutions per minute.

The main shaft must be sufficiently rigid so that it will not vibrate or whip. Runners become clogged with logs or other débris; and if this occurs on one side of the runner only, the entire unit may be thrown out of balance. To safeguard against serious accidents in this event, the main shaft should have an ample margin of strength. Where small shafts under high speed are employed, the matter of critical speed under runaway conditions should be carefully investigated, and, if necessary, additional intermediate steady bearings should be provided so that under maximum runaway the critical speed for the largest span will be at least twice the runaway. The following formula for critical speed may be used:

$$R' = \frac{30{,}700d}{L^2}, \qquad (170)$$

where R' = critical speed in revolutions per minute;
d = shaft diameter, in inches;
L = span between bearings, in feet.

It has been found that forged open-hearth steel shafts of approximately the following chemical and physical properties give excellent results.

Physical	
Ultimate strength	60,000–75,000 lb. per sq. in.
Elastic limit	30,000–42,000 lb. per sq. in.
Elongation in 2 in.	about 25%
Reduction in area	about 35%

Chemical	
Carbon	.25 to .35
Sulphur, not over	.045
Phosphorus, not over	.045
Silicon	.02
Manganese	.40 to .60

While high-carbon-steel shafts and special chrome steels may have greater physical characteristics, they have not been found entirely satisfactory under

the conditions to which they are subject. Hydraulic turbine shafts are frequently subject to shock caused by unstable conditions, and for this reason a fairly soft ductile material should be used. Large-sized shafts should be hollow-bored and inspected for flaws. All important shafts should have test bars and specimens taken from both ends. Hollow-forged shafts are usually of more uniform material, but up to the present time their excessive cost has not warranted their general use.

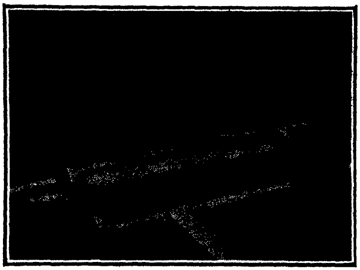


FIG. 404.—Main Shafts for 70,000 hp. Niagara Falls Turbine, 34 In. Diameter, Forged Flange on Both Ends, 6-in. Hollow Bored, Split Steel Sleeve where Shaft Passes through Bearing.

There are several different methods of connection between turbine and generator shafts and between turbine shaft and turbine runner. The shafts on the small units of the early days were usually of cold-rolled steel, and separate flanges were pressed and keyed on to the end. The runner was usually pressed on a straight fit on the lower end of the shaft where it was keyed and locked. At the present time there are two approved methods of attaching runners to shafts. The most generally used method is a taper fit, the runner being pressed on to the tapered end of the main shaft where it is securely keyed and then prevented from moving back on the taper by a split stop ring which fits into an annular groove on the main shaft.

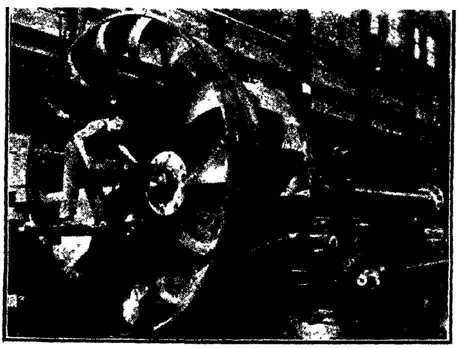


FIG. 405.—Fitting Key on 170-in. Discharge Diameter Runner, Rated 24,000 hp. at 70-ft. Head. Runner is Pressed onto Tapered End of Main Shaft, Key is Inserted, then Stop Ring is Inserted in Groove in End of Shaft to Prevent Endwise Motion. Cap encloses end of shaft and holds stop ring in place. (Allis-Chalmers Mfg. Co.)

Figure 404 shows a number of water-wheel shafts which have forged flanges at both ends, one end for coupling to generator flange, the other end for bolting to runner. It will be observed that the larger shaft is provided with a steel sleeve around it which is split and is held around the shaft by shrink links. A key, similar to that shown on the small shaft in the foreground, is provided to prevent the sleeve turning on the shaft.

Figure 405 shows the method of fitting keys where the runner is pressed onto a tapered main shaft. In this case two keys per slot are used, these keys being tapered to provide a wedging action. After the key has been driven,

the stop ring is fitted into the shaft shown just below the end of the runner fit, and then a cone or tip is bolted to the runner, enclosing the end of the shaft and preventing the stop ring from working out.

When the main turbine bearings are of the babbited oil-lubricated type, the main shaft is allowed to come in direct contact with the bearing surface, but when the water-lubricated lignum-vitae type main bearings are used it is excellent practice to provide either a steel or bronze sleeve around the shaft in order to take the wear. This enables replacement to be made without weakening the shaft. Considerable wear is likely to take place in lignum-vitae

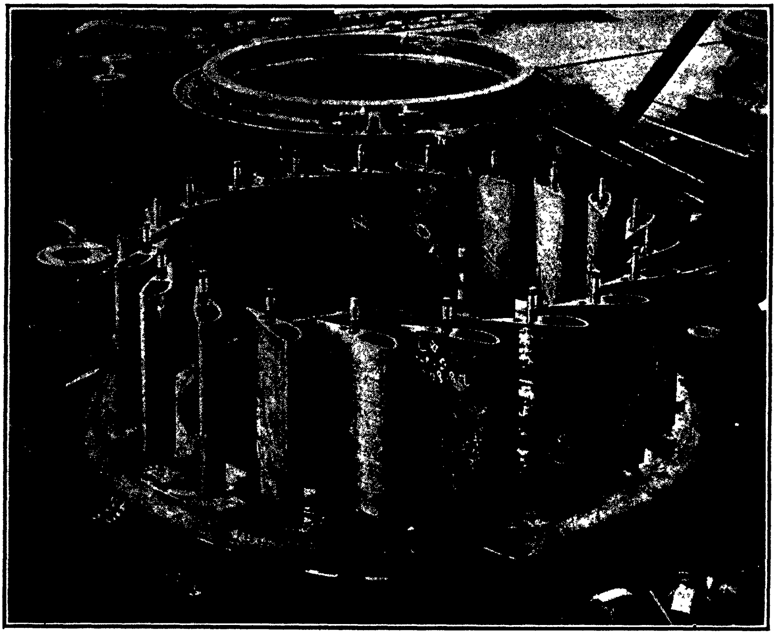


FIG. 406.—Guide Vanes and Mechanism for Open-flume Turbine. Guide vanes turn on Bolts Screwed into Discharge Ring, and are Operated by Links Connecting to Shifting Ring which rests on discharge ring. Guide vanes shown about 50 per cent open.

bearings, especially if there is any sediment or sand in the water. This has not been the case, however, in oil-lubricated babbit-type bearings where there is very little danger of wear on the shaft. For making the sleeve, steel has been found to be a much better material than bronze as it will stand the rubbing action and show much less wear. These sleeves may be made in one piece and shrunk on, or they may be split axially and held on with shrink links.

324. Guide Vanes.—Guide vanes for both Francis and propeller-type runners are of two general types: the open-flume or inside type, where the connecting links are exposed to the flow of the water; and the spiral-cased or outside type where the upper guide-vane stems extend up through the cover

plate and the operating mechanism is located on the cover plate outside of the water. The open-flume type guide vanes are usually made of cast iron, although recently some manufacturers have made them of plate steel, the plates being formed and welded. The guide vanes for outside gate mechanism are usually constructed of cast steel. These have the one pivot extended, and for this reason cast iron is not sufficiently strong as the extended pivot must transmit the full torque to the guide vanes.

Figure 406 shows the arrangement of guide vanes for an open-flume turbine. The guide-vane bolts or pivots are screwed into the discharge ring at the bottom, and the upper end extends up through the cover plate, which has been removed for this photo. These guide vanes are of cast iron with bronze bushings pressed in to form bearings. Recently guide vanes have been built up by forming sections of plate around dies, welding the edge where the plate ends meet, and welding steel blocks in to form the bearings. Fig. 406 shows the arrangement of links and the movable shifting ring located on the discharge ring, for moving the guide vanes. This type of mechanism is known as "inside type" because the links and pivots are inside of the wheel pit and exposed to the water.

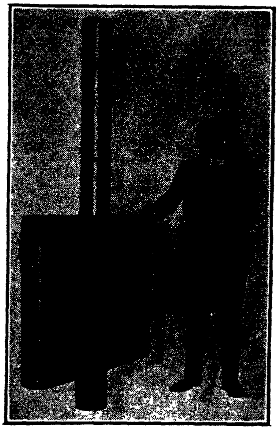


FIG. 407.—Cast-steel Guide Vane for Turbine having Outside Gate Mechanism. Upper stem extends through cover plate and connects to shifting ring through lever and link. For 70,000 hp. Niagara Falls turbine.

Figure 407 shows a large cast-steel guide vane of the outside gate type, such as is used with cast spiral, steel-plate spiral, and concrete spiral-cased turbines. This guide vane is used in the turbine shown in Fig. 381. The lower pivot is carried in a bronze bushing in the lower guide-vane ring, which is lubricated from the top by forcing grease through a hole bored through the stem. The upper pivot extends through two bearings in the cover plate, and an adjustable packing box is provided between these bearings to prevent leakage along the stems. A lever is keyed to the pivot above the upper bearing and is connected to the shifting ring with short links which are designed to break if some obstruction clogs the gates. The connection of levers, links, and shifting ring is shown in Fig. 385.

The customary type of guide-vane operating mechanism for the open-flume turbines consists of a shifting ring located either above or below the guide vanes, the ring being mounted on pads so that it can be rotated slightly, connection being provided from lugs on the shifting ring to an operating shaft which connects to the governor. Short links are used to connect the shifting ring to the guide vanes. These links are not made with a breaking section. A refinement in this design is to make one of the pins slightly eccentric, so that some adjustment can be made and the gates closed to decrease leakage. The same is true of the outside gate mechanism, especially for medium and

high-head turbines, where leakage during shut-down periods may cause considerable damage to the turbine parts. In case some obstruction clogs the vanes, even though they are provided with breaking links, there is likely to be some distortion to the guide-vane lever or stem. In order to obtain a tight closing fit, if the turbine is not provided with eccentric adjusting pins, it would

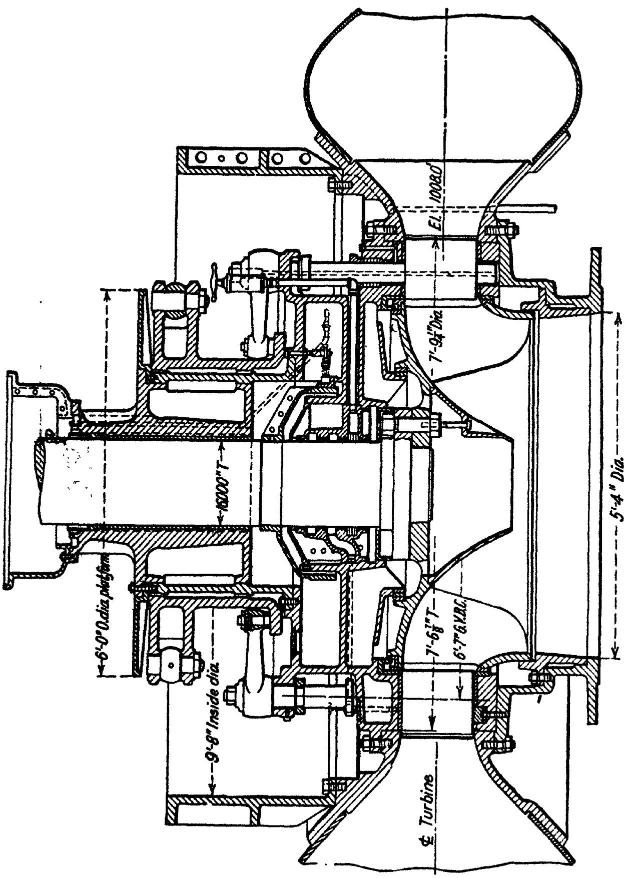


FIG. 408.—Section through Plate Steel Spiral Cased Turbine for 370-ft. Head, showing Renewable Liner Below Runner, Renewable Wearing Rings at Runner Clearances and Facing Plates in Guide Case. Bronze bushed individually lubricated guide vane pivot bearings, one below water passage, two above with adjustable stuffing box between upper two bearings. Thrust bearing to take upward thrust on guide vane stems. Cover plate of double construction drained through runner at center. Babbitted oil lubricated steady bearing with oil reservoir and viscosity pump in cover plate. Babbitted grease lubricated stuffing box with lignum vitæ seal blocks and drains to prevent high tail-water entering oil reservoir. Shifting ring slides on bronze pads and connects to guide vanes through breaking links, eccentric adjusting pins and long levers keyed to guide vane stems.

be necessary to fit the levers to the guide vanes with offset keys, which means a long shut-down.

325. Cover Plates. Cover plates, on open-flume turbines are usually made of cast iron and can be very simple in construction, the principal requirement being that they be sufficiently rigid to hold the proper alinement of the guide vanes. Sometimes staybolts from the discharge ring are used to add rigidity, but probably the best method is to use struts or braces from the flume walls as shown in Fig. 375. This is necessary because when the water

passes through the guide vanes and is accelerated, it exerts a backward force upon the guide vanes. The discharge ring of the turbine, being attached directly to the foundations, can carry its share quite readily; but unless the cover plate is supported rigidly it will tend to rotate slightly, setting up vibrations which have been known to break off the guide-vane stems. Under these conditions the most satisfactory support is from the flume walls rather than with staybolts.

Cover plates of concrete spiral-cased and plate-steel casing turbines are more elaborately constructed. The bearings for the guide-vane stems, where they extend through the cover plates, should be bronze bushed and some provision should be made to carry away whatever water leaks up along the guide-vane stem. Sometimes the cover plate is designed so that any leakage water through the lower bearing is drained over and down into the draft tube through the cored openings in the runner crown. The cover plate shown in Fig. 408 is of a materially better construction. It has two bearings above the guide vanes, and between these two bearings there is provided an adjustable packing box so that all leakage along the bearing can be eliminated. This cover plate and runner are designed with labyrinth seals to decrease the leakage past the runner clearance. A cored passage is also provided in the cover plate to lead this water over to the drain holes in the center of the runner. This is known as a "double deck" cover plate. It prevents the building up of pressure on the runner crown and decreases the hydraulic thrust 20 or 25 per cent.

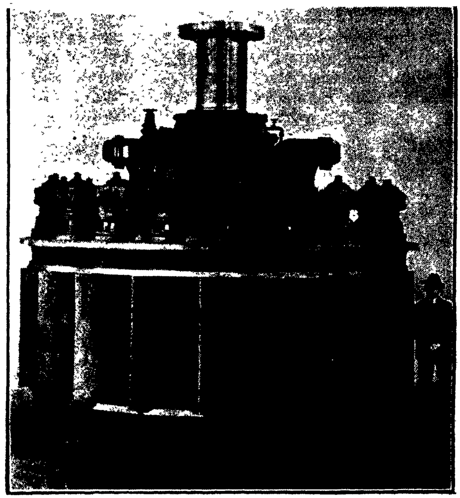


Fig. 409.—Working Parts of Concrete Spiral-cased Francis Type Turbine for 80-ft. Head, 17,500 hp. 65 Specific Speed, showing Cast Steel Guide Vanes with Operating Mechanism on Cover Plate, Links Connecting to Shifting Ring and Twin Strainer in Water Supply to Lignum Vitæ Main Shaft Steady Bearing. (S. Morgan Smith Co.)

Figure 409 shows another type of cover plate which has a detachable cage bolted into it, these cages combining the adjustable packing box and the upper guide-vane bearing. This design also shows the guide-vane levers which, in this case, are clamped on to the stems. The bolt and cap above the lever provide for vertical adjustment of the guide vanes so that they do not rub on the guide case.

Figure 410 shows a comparison between two types of guide-vane bearing and stuffing box, the one at the left being the most substantial construction, with adjustable packing box between two bearings. The one at the right has

proved very unsatisfactory, as with this construction it was impossible to keep the water from leaking out on to the top of the cover plate.

326. Wearing Rings and Facing Plates.—Wearing rings and facing plates have renewable steel or bronze surfaces. They are provided where it is desirable to maintain small clearances in order that leakage may be held down to a minimum. Facing plates are used to line the upper and lower surfaces of the guide case adjacent to the ends of the guide vanes (see Fig. 408). They are usually constructed of either steel or bronze plates from $\frac{3}{8}$ to $\frac{3}{4}$ in. in thickness, made either in sections or in continuous rings, and are used on a majority of large turbines for heads above 75 ft. When the turbine is shut down and the guide vanes are closed, the water tends to leak past the ends of the guide vanes, and if it contains sand or foreign matter, it quickly wears away the surface of the guide case. As higher heads and larger-capacity turbines are used, it is important to decrease this leakage and the machines must be designed so that they can be quickly restored to their original condition.

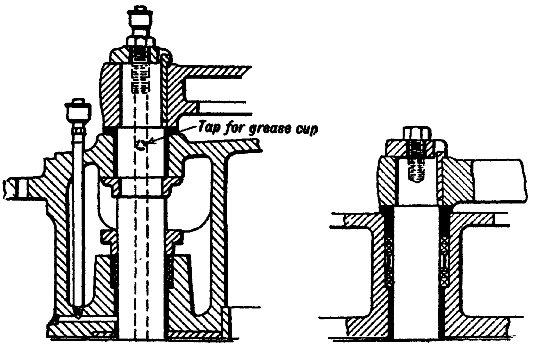


FIG. 410.—Alternative Types of Guide Vane and Cover Plate Design, the One at the Left Having Two Bearings with Adjustable Packing between, the Other Having Packing Forced in between Two Bushings.

Wearing rings are used on the clearances adjacent to the runner band and crown. They are distinctly shown on Fig. 408. The leakage past the clearance of medium and high-specific-speed runners is not serious, but for low-specific-speed runners under high heads the leakage through the runner clearances may seriously affect the efficiency. Sand and foreign matter in the water rapidly wear these clearances, especially under heads above 200 ft. With runners below 30 specific speed, the leakage through these clearances, even in their original condition, sometimes amounts to as much as 3 or 4 per cent, so that any wear will greatly increase this loss. Labyrinth seals, that is, seals formed by several stages of wearing rings, reduce this leakage about in proportion to the number of stages.

Recently it has been common practice to omit the wearing rings on the runner band, but to provide them on the stationary surface of the discharge ring and cover rings, since they may be renewed. This construction is shown in Fig. 387. Experience has shown that only in a few cases have the wearing rings been renewed on the runner proper, since it is usually found that the runner itself is so worn that it does not pay to repair it, a new runner being more economical.

327. Bearings.—Turbine bearings for vertical units are of two principal

types. Those for water lubrication (Figs. 377, 381 or 384) are usually of lignum-vitae or a similar wood, and those for oil lubrication (Fig. 408) are of the babbited type. Water-lubricated lignum-vitae bearings are used on a majority of installations, both the open-flume type and the enclosed spiral-cased turbines of the vertical type. Where clean water is available or where it is possible to install a settling tank, bearings of this type last many years without renewal; but where there is grit in the water or where, during flood times, only dirty water is available, they become worn quickly, sometimes allowing the runner to rub. On the small open-flume turbines, the lignum-vitae bearings are usually constructed by simply setting four blocks of lignum-vitae in pockets, cast in an iron bearing shell, adjusting screws being provided to press the blocks against the shaft. This type is readily adjustable. In the larger sizes strips of lignum-vitae about 2 in. square are set into dovetail grooves, cast into the bearing shell. These bearings are not adjustable. When they become worn they must be relined and rebored. Adjustable bearings of the latter type have been developed, however, where sections or shoes of cast iron containing the dovetailed grooves are set in an outer housing so that they may be adjusted when the bearing wears. These have many advantages.

When the bearing is operated submerged, as in the open-flume construction, no other lubrication is required, although sometimes grease is used in addition. This applies both to vertical-shaft and horizontal-shaft units. In the encased-type units, provision is made to supply a steady stream of water to the bearing, and grease is usually supplied in addition. Some type of strainer is used for straining this water supply, and it should be in duplicate so that one strainer may be cleaned while the other one is in use. This is desirable with this type of bearing, especially in cold climates where there is danger of the strainer being clogged up with fine ice, allowing the bearings to run dry. In some plants this has been remedied by running the water through a traveling screen, and in some places the water is run through warming tanks before entering the bearing.

Oil-lubricated bearings of the horizontal type, as shown in Figs. 387 and 388, have been in general use for a long time, but the vertical type of oil-lubricated bearing has not been used extensively. There are, however, at the present time, a considerable number of installations of the vertical-shaft units with oil-lubricated turbine bearings, as shown in Fig. 408, which are operating very satisfactorily. Some of these have a reservoir just below the bearing, located in the cover plate, and one or two pumps driven from the shaft, which simply lift the oil to the top of the bearing and allow it to return through the bearing. In other installations, the oil is circulated through an external tank by a motor-driven pump. The latter system, however, is not as compact and is not as reliable as the self-contained system.

Probably the reason why vertical oil-lubricated bearings are not more popular is because some engineers fear that the oil will be lost on account of the suction down along the shaft. With the proper design, this is not the case. If a good oil slinger is provided below the bearing, and a suitable stuffing box with ample drain area, there is no danger of losing oil. Bearings have

been in operation for several years without appreciable loss of oil. In case of high tail-water, there is some danger of the bearing being flooded out, if the unit is shut down, but this can also be prevented by a suitable packing box and drain below the bearing. If the unit is in operation, the vacuum below the runner is usually sufficient to prevent any water rising along the shaft.

The horizontal bearings are usually of the ring-oiled pedestal type, similar to those used on generators, and have proved very satisfactory. Usually the circulation of oil from the rings is ample, although some exceptionally large bearings have been designed to use oil under pressure when starting up.

328. Speed Rings.—The speed ring is usually considered that part of the turbine which connects the cover plate and the discharge ring. In its simplest construction it consists of vertical ribs attached to an upper and lower ring. These ribs carry the load from above or take the strain of the internal water pressure. There are three distinct types of speed rings: those for concrete spiral-cased units, those for plate-steel spiral-cased units, and those for cast-iron or cast-steel casings. Recent practice has been to cast the speed ring and ribs integral with the cast-iron or cast-steel casing, as shown in Fig. 384, this resulting in a much simpler and better construction if the casting is properly made. These ribs for the vertical unit must be sufficiently strong in tension to take the entire internal pressure from the water, and they must be designed so that in compression they will carry the load from above the casing. This usually includes the mass of concrete, the generator, and the weight of the revolving parts, including runner, shaft, and hydraulic thrust. For medium and low head, with cast-iron or cast-steel cased units, the speed-ring ribs are sometimes made of turned bolts, the casing simply being cast open on the inner diameter, and these bolts are threaded into both flanges.

Speed rings for plate-steel spiral-cased turbines are usually of cast steel, the ribs and flanges being cast together, the flanges extended and flared so that the plates may be riveted to them, as shown in Figs. 381 and 408. Cast iron is not considered suitable for this type of speed ring, because it may be cracked in riveting, and while these cracks may not show up immediately they are likely to appear afterwards.

Speed rings for concrete spiral-cased units for low heads take a variety of forms, the most common of which are as follows:

Cast-iron speed rings, as shown in Fig. 402, are very similar to those used with plate-steel castings, having the ribs and flanges cast together, these sometimes being designed to have the reinforcing rods attached to the flanges as the loads that they carry are very similar to those carried in the plate-steel cased units. For this purpose, with large units under relatively high heads, cast-steel rings are sometimes used.

Another construction, the staybolt type, is to make the upper and lower rings of cast iron and make the ribs of steel rods, these usually being turned and threaded for bolting into the upper and lower flanges.

The simplest construction of all is to make only the ribs of cast iron, flaring these at the top and bottom and embedding them in the concrete, relying entirely on the concrete for the upper and lower surfaces. With this con-

struction, the ribs have no connection with the other turbine parts except as they are tied into the concrete. Under some conditions, only the lower ends of the ribs are embedded in the concrete, as shown in Fig. 376, the upper ends being bolted to a flange constructed of plate steel built up of structural-steel sections.

With the more common type of speed ring, the pit liner, cover plate, discharge ring and guide-vane rings are bolted to the speed ring, although with the speed ring used in concrete cased units, the discharge ring is frequently omitted, the lower flange of the speed ring serving this purpose as shown in Fig. 377. When the cast ribs are used, with a plate-steel upper flange, this upper flange also is designed as a pit liner.

When the diameter of the speed ring is such that it cannot be shipped in one piece, the sections are bolted together and sometimes additional shrink rings are used, especially for large plate-steel units.

329. Casings.—Casings are of four principal types: open-flume casings, concrete spiral casings, plate-steel spiral casings, and either cast-steel or cast-iron spiral casings. The open-flume casing, in its simplest form, is simply a square chamber in which the turbine proper is set, Fig. 375, showing this arrangement. Sometimes the downstream wall is made circular. In the concrete spiral casing, there are many forms that have been used, the different forms having certain advantages for certain special conditions. Fig. 378 shows a form of concrete casing where the nose of the casing is on the downstream side. A design, however, which is probably the most economical as regards spacing and simplicity of construction, is that where the nose piece is located on the center line either at the right or left side, depending on the direction of rotation.

The low-head open-flume or concrete spiral casings are comparatively simple in design as the concrete is usually sufficiently strong in itself, only a slight amount of reinforcement being required. However, for 60 or 70-ft. head, the matter of reinforcing the concrete casing is a difficult problem, and careful study is required in order to arrange the reinforcing bars in the most advantageous manner. Concrete casings for these heads usually approach very closely the form of plate-steel or cast casings.

Plate-steel casings are constructed by riveting sections of plate to the flanges of the speed ring, successive sections of plate being of reduced diameter so that the area of the spiral casing decreases as the water travels around the turbine. Figs. 381 and 382 show this construction quite clearly. This design is used extensively for medium heads and for both large and medium-sized units. The plates are usually punched and rolled and then fitted together in the shops of the manufacturer, together with the speed ring, the holes for the rivets all being reamed and the plates then carefully match-marked so that they may be readily assembled in the field, where they are riveted and calked. The joints are either single or double riveted, depending on the pressure, and are usually lapped, although butt joints with butt straps outside are used by some manufacturers. On account of the decreasing diameter, the plates may be of lighter metal at the smaller end of the casing.

Cast-steel casings are especially suitable for high-head installations,

although the casing should be designed sufficiently thick in order to insure excessive strength in case of a poor-quality casting, since it is impossible to inspect the material of a casting for internal flaws. While cast-iron casings are used quite extensively, sometimes for relatively high heads, they are not considered as suitable as cast steel, because of the brittle structure of cast iron and because it is not strong in tension. There are many cases on record where cast-iron casings have broken on account of water shock, although there are few, if any, cases where cast-steel casings have broken. In large sizes it is practically impossible to make a steel casting less than $1\frac{1}{2}$ in. in thickness;

FIG. 411.—Assembled View of 35,000 hp. 850-ft. Head, 21 Specific Speed High-head Cast-steel Spiral-cased Turbine for Portland Railway, Oak Grove Dev. Butterfly valve, governor-operated pressure regulator, servo motors bolted to pads on casing, main steady bearing of oil-lubricated babbitted-type with auxiliary motor driven pump mounted above bearing. (Pelton Waterwheel Co.)

therefore the ability of the foundry to cast the section may require greater thickness than is necessary for strength alone. Fig. 411 shows a cast-steel casing for a 35,000-h.p. unit under 850-ft. head. This is probably the highest-head turbine of large capacity in the world, and the design of the casing required special care. It is made in halves, and the speed-ring ribs are cast integral with the casing. In the cast-iron casing shown in Fig. 412, the speed ring is cast separately and the cast casing is bolted to the speed-ring flanges. This casing is also made in halves.

330. Regulating Connections.—The regulating connections on small turbines of the spiral-casing and open-flume type usually consist of a regulating shaft with push and pull rods attached to opposite sides of the shifting ring.

In the vertical open-flume units the regulating shaft extends up through the generator floor and a bearing is usually provided in the governor base as shown in Figs. 417 and 419, the governor levers being pressed and keyed on to the shaft, and the governor forces being exerted through this lever. The lower end of the shaft usually rests in a step bearing attached either to the flume wall or to the flume floor, and a double lever is keyed to the shaft in line with the shifting ring. Two adjustable push and pull rods connect the double lever, one to each side of the shifting ring. These should be adjustable; all the moving parts should be bronze-bushed and provision should be made for forcing grease into these connections whenever the flume is drained. Probably the

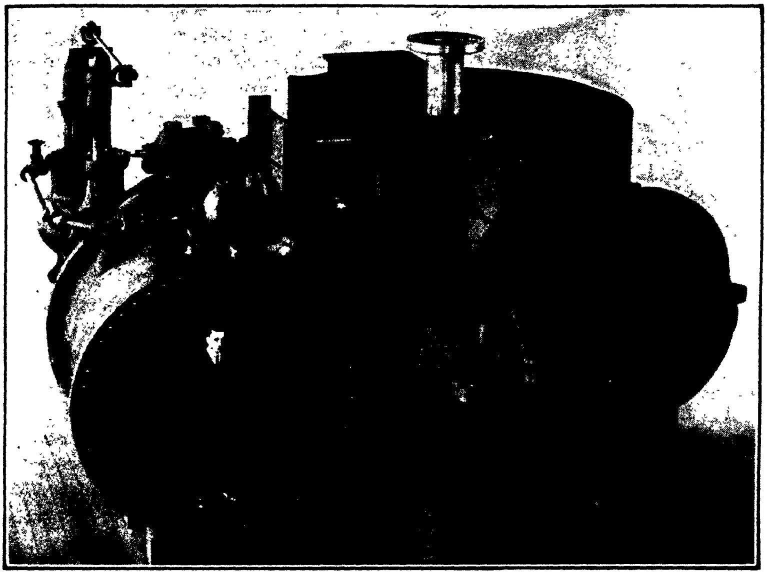


Fig. 412.—Cast-iron Spiral-cased Turbine, Casing Made in Halves with Separate Speed Ring. Cast-iron pit liner and servo motors bolted to pads on casing, pressure regulator governor operated. (S. Morgan Smith Co.)

best arrangement is to carry the weight of the regulating shaft and connections from a thrust surface on the governor base, as this is not exposed to the water, is accessible, and can be kept properly greased, whereas if the weight of the shaft is carried in the step bearing it is not always convenient to keep this thoroughly greased and wear may result. Horizontal units use practically the same type of regulating mechanism, the regulating shaft running horizontal, especially in the case of the twin or quadruplex horizontal units. The regulating shaft runs the full length of the unit, and double levers and push and pull rods are provided opposite each shifting ring as shown in Fig. 386.

For some spiral-casing units of medium size, such as shown on Fig. 376, the same type of regulating connections are used even though the guide-vane

mechanism be of the outside type. It has been found that for units requiring a governor not exceeding 25,000 or 30,000 ft.-lb. capacity, the regulating-shaft mechanism described above is the most economical; but when the governor capacity exceeds approximately 30,000 ft.-lb. it is usually more economical to locate the regulating cylinders in the turbine pit and connect one piston rod each side of the shifting ring. The customary construction is shown in Figs. 381 and 411. This arrangement makes a positive and direct connection, practically equal forces being exerted on both sides of the shifting-ring surface. Some provision must be made for supporting these regulating cylinders rigidly, and usually a cast-iron barrel or ring is provided to which they are bolted, although a casing of the cast-iron type may be bolted directly to the top of the casing as shown in Fig. 412.

331. Draft-tube Liners.—Except on relatively small units under low heads, it is desirable that at least the upper portion of the draft tube be lined with either a cast- or plate-steel liner, as the water discharging from the runner has a rapid whirling motion, and where unlined draft tubes have been used there have been frequent cases where the concrete has been cut away, to a depth of 12 to 18 in., so that it seriously decreased the strength and endangered the fastening of the discharge ring and other turbine parts. The resistance of concrete to disturbances of this kind is low, and it has been found that a light plate-steel liner prevents these troubles.

Draft-tube liners can be readily made of plate steel riveted together. They should be provided with flanges at top and bottom, and anchor straps, so that they will get a good bond with the concrete. Either a cast- or structural-steel flange should be used at the top, and this should be bolted to the turbine proper.

On high-head units where the velocity in the draft tube may be high and where the upper section of the draft tube is exposed, in order to permit removal of the runner from underneath or for inspection purposes, this upper section of draft tube is usually made of cast iron and man-holes should be provided on opposite sides to permit inspection. An interesting design of draft-tube top is of the telescopic type, shown in Fig. 383, where this upper cast-iron section is so arranged that it may be lowered directly into the lower part of the draft tube, thus leaving a clear space for inspection purposes when in the lowered position. This telescoping section is bolted to the bottom of the discharge ring, and a packed joint is used at the lower joint to prevent leakage of air into the draft tube. This and similar constructions have been used extensively and have resulted in a material saving in time, especially where frequent changes of the runner were required, such as might be due to silty or bad water conditions.

The design of the draft tube has probably received more publicity during the last few years than any other single part of hydraulic turbines. It has been proved conclusively that the concentric type of tube, as shown in Figs. 377 and 381, is the most efficient, considering the space required, although the characteristics and specific speed of the runner determine largely the relative importance of draft-tube efficiency. A large number of draft-tube tests were conducted at the Worcester Polytechnic Institute (see A. S. C. E.,

Vol. LXXXVII, page 893) with about a 75 specific speed runner. The efficiency of the runner with no draft tube at all was about 85 per cent. The maximum efficiency obtained with the concentric type of draft tube was 88 per cent, whereas some poor type of elbow draft tube actually decreased the efficiency below that obtained with no draft tube at all. The elbow type of draft tube shown in Figs. 380 and 383 gives efficiency within about ¾ of 1 per cent of that obtained with the best type of concentric tube for runners having a specific speed of about 75 or below. The concentric type of draft tube requires greater width than the elbow type, so that the saving in power-house length by using the elbow tube must be balanced against the power lost by the slightly decreased efficiency.

With high-speed runners having a specific speed of 125 to 175, the amount of energy in the water discharged from the runner is much greater, and usually the concentric type of draft tube has been proved conclusively to be the more efficient for this type of runner, since the whirling component of the water is very large. Tests have shown that there is an improvement of at least 3 per cent in the efficiency of the unit when used with the draft tube shown in Fig. 381, as compared with the tube shown in Fig. 380. In addition to the difference in the efficiency of the unit, the power output is affected by a greater percentage than the efficiency. With the low-head installations with which the high-specific-speed runners are used, it is generally possible to install the concentric tube, although it requires a greater width, because this greater width is also advantageous in producing slow velocity in the casing leading to the unit. Concentric-type draft tubes have been used with practically all the high-speed runner installations in this country.

The elbow type of draft tube is usually made in concrete, the upper part being provided with a liner, but the concentric or " Hydraucone " type may be made either in concrete or plate steel, or concrete lined with plate steel. The formwork of the concentric tube is relatively complicated, and a material saving usually can be made by the use of a complete plate-steel tube as shown in Fig. 375.

332. Impulse Wheels.—The design of an impulse wheel does not include as many variables as are encountered in computations of Francis-type runners. Specific speeds of 3.0 to 4.5 are ordinarily used for large units, the particular value lying between these depending largely on the manufacturer, the use of developed sizes, and the selection of economical and suitable synchronous speeds for generators. For instance, the 14,000-h.p., 1750-ft. head, 300-r.p.m., single overhung unit shown in Fig. 388 has a specific speed of 3.14. A speed of 360 r.p.m., giving a specific speed of 3.76, might have been used for this particular unit, but it is planned to install additional units of the double overhung type, 28,000 h.p. per unit, each wheel end being a duplicate of the present 14,000-h.p. units, and since a speed of 360 r.p.m. and 28,000 h.p. would result in an uneconomically long generator the speed of 300 r.p.m. was chosen for all of the units.

The quantity of water used by this unit to develop approximately 14,000 h.p. at 85 per cent efficiency is 83 c.f.s. Since the efficiency of the jet is about 97½ per cent the velocity of the jet may be assumed to be 97½ per cent of the

spouting velocity at 1750-ft. head, or 327 ft. per second. To discharge 83 c.f.s. will require a jet of 6.83 in. diameter. This wheel was built for a maximum jet of 7 in. diameter.

For best efficiency the impulse wheel should run at such a velocity that the point on the bucket at which the center of the jet strikes is traveling at approximately 44 per cent of the spouting velocity. This diameter, which is measured where the jet comes tangent to a circle having its axis at the center of rotation, is called the impulse diameter. To run at 44 per cent of the spouting velocity, or 148 ft. per second for 300 r.p.m. will require a wheel of 113 in. impulse diameter. This wheel as built had an impulse diameter of 115 in. This gives a ratio of impulse diameter over jet diameter of 16.45 to 1. For ranges of specific speed between 3.0 and 4.5, this ratio varies from 11 to 17.

Impulse buckets having different designs are used by different manufacturers and they are rated according to different dimensions, some manufacturers using approximately the same size bucket as the jet diameter or slightly smaller. On an average the bucket diameter is from 3 to 3.5 times as wide as the jet, that is, for a 7-in. jet the bucket width will be between 21 in. and 24.5 in. The buckets shown in Fig. 388 are 22.75 in. wide or 3.25 times the jet diameter. The maximum or over-all diameter of the impulse wheel, that is, the disk with its buckets, is considerably larger than the impulse diameter, since the jet strikes at approximately the center of the bucket, the wheel in Fig. 388 being $131\frac{1}{2}$ in. diameter or about 14 per cent larger than the impulse diameter. Impulse wheels hold up their efficiency very well for either over-speed or under-speed, 10 per cent either over or under normal speed resulting in less than 1 per cent decrease in efficiency. This corresponds to a peripheral coefficient, or ratio of wheel velocity to spouting velocity, of from 40 to 48 per cent.

Unlike Francis and propeller runners, impulse wheels do not require different specific speeds for different heads, for reasons of pitting or strength, although a small-capacity wheel under a very high head may result in too high a speed for economical generator construction unless a very low specific speed is used. Specific speeds as low as 1.1 have been used to obtain reasonably low speeds for small units under high heads, and specific speeds as high as 7 have been used for medium capacities under very low heads, to give higher speeds for economical generator construction.

Impulse wheels in this country are practically all of the horizontal type with only one jet for each disk. Frequently two disks, one on each end of the shaft with the generator in between, have been used. This type of unit, as shown in Fig. 389, usually consists of a single disk, with from 18 to 24 buckets mounted around the periphery and with one jet directed tangentially on the buckets, usually horizontally below the shaft. A housing, either of plate steel, cast iron or concrete, or a combination of these, encloses the disk, some provision being made to prevent leakage of water where the shaft enters the housing. The bearings which support the shaft of the impulse wheel are the customary horizontal bearing of the ring-oiling pedestal type, similar to the standard type generator bearing.

Figure 413 shows a forged-steel disk having twenty buckets mounted on it, each bucket being held with three fitted bolts. An interesting type of construction is the chain bolting, where the bucket lugs are interlocked and one bolt serves to hold the forward lug of one bucket and the rear lug of the next bucket. Small impulse wheels are sometimes made with the buckets cast integral with the disk, but this has only been found practical for small-sized units such as those used to drive oil pumps, exciters, and small service units. Cast steel is used almost exclusively for the material for large buckets, although cast iron has been used for low-head installations and bronze is used on small units on account of the greater ease with which it can be finished and fitted.

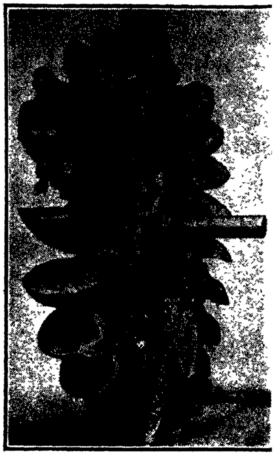


Fig. 413.—Buckets and Disk of Largest Impulse Wheel in America, 34-in. Cast Steel Buckets, 155-in. Impulse Diameter, 30,000 hp. 1008-ft. Head. (Allis-Chalmers Mfg. Co.)

The disk of the impulse wheel should be made of good-quality material. Forged steel, annealed, has been found satisfactory for large units, although cast-steel and cast-bronze disks have been used especially for small installations. Laminated disks, built of steel plates, also offer possibilities. However, the selection of the material for the disk should be given considerable care, as the runaway speed of the impulse unit is practically double the normal speed and dangerous stresses may be set up in the metal under these conditions.

On large impulse wheel installations it is economical to make the housing of plate steel, but this must be reinforced properly to prevent breaking and all the joints must be calked water-tight. Cast iron is an excellent material for small housings but the cost makes it almost prohibitive for large installations. In some cases the lower part of the housing has been formed directly in the concrete, but this should be lined with a metal liner, as jets of water under high heads will rapidly erode concrete surfaces.

The nozzle and needle pipes on impulse wheels for high-head conditions and large diameters should be of cast steel, although cast iron has been used for relatively low pressures. One type of needle and nozzle pipe is shown in Fig. 414. A removable needle tip and removable throat ring would be provided, so that in case of wear due to some foreign matter of silt in the water, these parts can be readily replaced, as worn needle tips and throat rings will seriously decrease the efficiency of the unit because they cause distortion of the jet so that it cannot operate efficiently on the buckets.

It is desirable to place the buckets as close to the nozzle as possible, as there is always a tendency for a jet of water to spray out. If the buckets can be kept close to the nozzle where the jet is still in practically perfect condition, the efficiency of the unit will be much better than if the buckets are at such a distance from the nozzle that the jet is allowed to distort. Usually the

needle which controls the size of the jet extends backward to the outside of the needle pipe, the needle pipe being curved or slightly " S " shaped to allow for this. The governor or mechanism for controlling the position of the needle may be attached to the needle stem through levers on the servo-motor, or a regulating cylinder may be mounted directly on the nozzle pipe and connected directly to the needle as shown in Fig. 414. For small installations the former is the accepted practice, although on large units, the direct-connected servo-motor has certain advantages. The pipe curvature and size of the nozzle pipe has a material effect on the efficiency of the jet and on the efficiency of the unit, and great care should be taken in the design, as high velocity, dis-

FIG. 414.—Nozzle Pipes and Needles with Auxiliary Relief Nozzles Below for 35,000 hp. Impulse Turbine. Governor servo motor connected direct to needle stem. (Pelton Waterwheel Co.)

turbances, or whirls caused by bends will greatly reduce the efficiency. A type of nozzle known as the " bifurcated nozzle " has shown excellent efficiency, the apparent reason being that the two streams of water, with an opposite tendency of whirl caused by the curves of the pipes, neutralize each other and result in a more perfect jet than that which issues from a single " S " curved pipe.

On account of the high heads under which impulse wheels usually operate, with resultant long penstocks, some form of pressure protection device is usually required, such as a pressure regulator, as surge tanks cannot be constructed economically for these heads. The location of the pressure regulator or relief valve varies. It should be located close to the needle, but its

outlet should be designed so that, under normal operating conditions, disturbances will not be set up in the flow of water to the jet, as these seriously affect the efficiency. One type of pressure regulator or relief valve is shown in Fig. 414, this being a second or lower nozzle attached directly to the nozzle pipe and of practically duplicate construction. This lower nozzle is designed to open when the upper nozzle closes, in order to maintain practically constant velocity in the penstock, then to close slowly so as not to set up serious pressure rises. Fig. 388 shows another type consisting of a standard pressure regulator which is attached some distance upstream on the nozzle pipe where the velocities are lower and the tendencies to cause disturbance are less, the

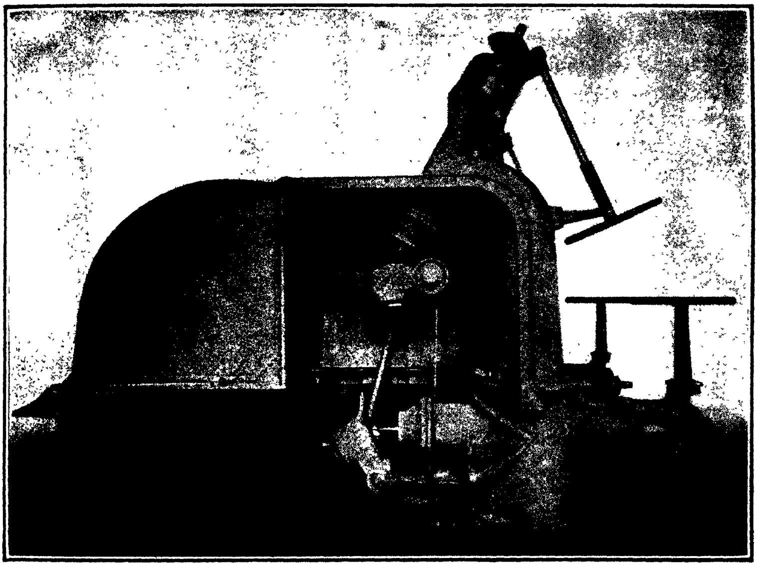


FIG. 415.—Two Jet Impulse Nozzles with Deflecting Hoods which Deflect Jet away from Buckets. Both needles hand controlled, governor sets on deflectors. (Pelton Waterwheel Co.)

operating characteristics of both being essentially similar. Both types are in satisfactory operation in a large number of installations. Other means of guarding against the pressure rise, which would be caused by rapid closing of the needle, are jet deflectors and rotating nozzles. In case of a change of load, the jet deflector cuts into the stream of water and throws part of it away from the buckets until the needle has readjusted the stream by slow movement. This device is shown in Fig. 415. The deflecting nozzle performs practically the same function, by tilting the nozzle pipe so that part of the jet misses the bucket, the latter scheme having the disadvantage that the moving joint in the nozzle pipe is hard to keep packed under high pressure. The former scheme, under some conditions may result in a distortion of the jet

which may cause pitting or wear of the buckets. It is felt that the design with separate relief valves is preferable to the latter types, although more expensive.

European designers frequently use more than one jet on a horizontal-shaft impulse wheel, as this results in a considerable saving in cost, since practically double the power is obtained from the same disk, but the crowded conditions and inefficient nozzle pipe used with this type of construction have caused a considerable decrease in efficiency and have made this type unpopular in this country. However, by proper design of the inlet pipes, avoiding abrupt curves, and with proper spacing of the nozzle to prevent the discharge from one interfering with the jet of the other, efficiencies comparable with those of the single jet may be obtained.

It is felt that as impulse units become larger, considerable saving may result by using a multiple-jet vertical-shaft unit. Some attempts were made at this construction several years ago, but the design was not good and consequently the results were not good. Tests, however, have demonstrated vertical shaft units, as shown in Fig. 390 with two, three, and four jets, may be designed to give efficiencies comparable with the single-jet unit and at a considerably lower cost per horse power, and it may be that the near future will see the development of large units of this type.

333. Governors and Governing.—Hydraulic-turbine governors are designed to regulate the speed of the unit by increasing or decreasing the amount of water supplied to the turbine in order to maintain a balance between water input and power output, acting when a change in power output causes a fluctuation in speed. The principal elements of governors are as follows:

1. The flyballs, or means of responding to speed change.
2. The distributing valve.
3. The regulating cylinders, or servo-motors.
4. The restoring or relay mechanism.
5. The oil or fluid supply under pressure.

Fundamentally, the action of a governor is as follows: A change in speed causes a change in the position of the rotating flyballs; this change in position moves the distributing valve so that oil under pressure is let into the regulating cylinders, causing them to move the gate mechanism of the turbine; this movement of the gates is transmitted back to the distributing valve through the restoring mechanism, bringing the distributor valve back to its original position after the gates have been moved an amount sufficient to compensate for the change in power output.

Flyballs.—The flyballs usually consist of two weights held together by springs, links being so connected that when centrifugal force separates the weights, the links serve to move the pilot valve. These flyballs are of a simple construction and are exactly similar to those used on steam engines, steam turbines, and other power units. Until recently, the accepted method of driving flyballs has been by belt from the main shaft, especially with horizontal-shaft units; or with bevel gears through a jack shaft, then with a belt from the jack shaft to the flyballs. A simplified arrangement is obtained

with the shaft or direct-connected flyball which is mounted directly on the main shaft. This flyball is shown in Fig. 416.

Another type uses the motor-driven flyball, shown in Fig. 417 which shows a Woodward type H. R. governor arranged for vertical regulating shaft. The motor which drives the flyball is distinctly shown in the upper right-hand corner of the picture, the motor being geared directly to the flyballs, thus doing away with all belts and shaft governor drives, the motor being operated

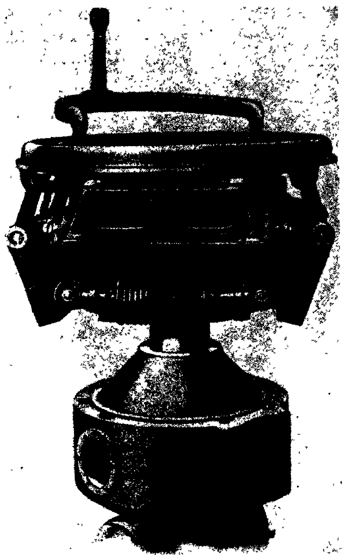


FIG. 416.

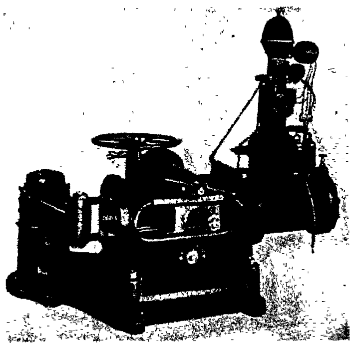


FIG. 417.

FIG. 416.—Direct-connected Flyball, built for Tennessee Power Co. (Allis-Chalmers Mfg. Co.)

FIG. 417.—Woodward Type H. R. Governor, 30,000 Ft.-lbs. Capacity Equipped with Motor-driven Flyballs, Synchronizing Motor and Separate Hand-gear Control with Vertical Regulating Shaft Extending through Governor Base where it Connects to Horizontal Cylinder through a Lever and Connecting Rods.

on 110-volt, three-phase alternating current from a transformer connected to the unit which it regulates.

Distributing Valve.—The distributing valve is usually of the balanced type, so that practically no energy is required to operate it, and its design should be such that when moved slightly it lets pressure into one side of the cylinder and connects the other side of the cylinder with the discharge, a slight movement in the opposite direction accomplishing the same results, but with pressure to the opposite side of the cylinder. On large-sized governors, two of these pilot or distributing valves are employed, one within the other, this

design being used to increase the sensitiveness, since large-sized governors require a rather large valve and this would take considerable energy from the flyballs to move it. The one pilot valve is located inside of the distributing valve, and the one follows the other almost simultaneously, so that there is no lost motion. Fig. 418 shows a section through this type of valve. The inner valve is operated directly by the flyball motion, a slight movement being sufficient to allow oil pressure to enter the chamber at one end of the larger or outside valve, which moves it an amount equal to the movement of the inner valve. The movement of the outer valve uncovers the ports, which admit oil pressure to one side of the regulating cylinder and allow oil to discharge from the opposite side, the resulting action being the same as though the inner and outer valves were locked together, but requiring much less energy to move them.

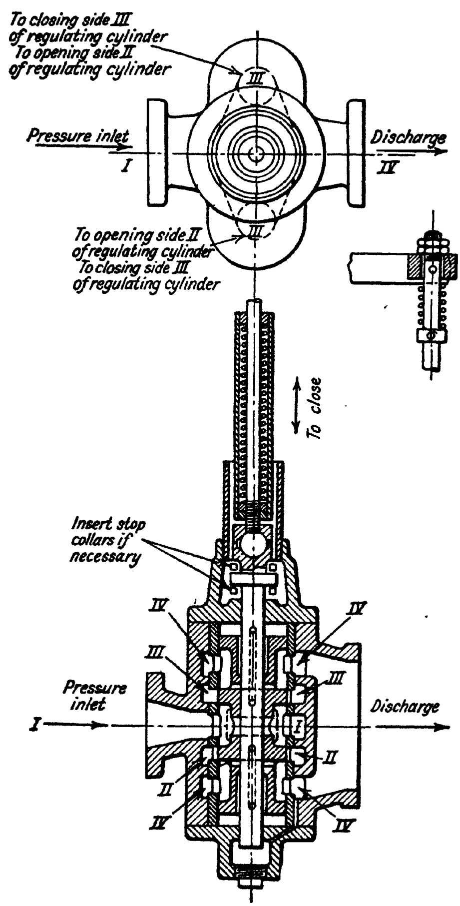


FIG. 418.—Section through Governor Regulating Valve Having Inner or Pilot Valve which Controls Position of Larger Distributing Valve. Pressure inlet at left, discharge outlet at right; space III connects to closing side of cylinder; space II connects to opening sides of cylinder.

Regulating Cylinders.—Regulating cylinders, or servo-motors, are located in the base of the governor for governors under 30,000 ft.-lb. capacity; but for units requiring larger governors, they are usually located in the turbine pit or mounted on the turbine cover plate, two being used in this case, one connected directly to each side of the shifting ring. In the governor shown in Fig. 419, the regulating cylinder is distinctly shown below the flyballs; the regulating shaft, which is vertical in this case, extends up through a bearing provided in the governor base where it is connected to the piston rod of the cylinder.

Figure 420 shows the governor stand for an actuator-type governor of 100,000 ft.-lb. capacity. The flyballs are mounted in the revolving case at

the top, the floating lever just below the flyballs operating the distributing valve enclosed in the housing at the left, the relay or restoring mechanism being connected to the floating lever between the flyballs and distributing valve. Gate opening is indicated by the scale at the top, some of the hand-control levers being shown below.

Relay Mechanism.—The relay mechanism, which brings the distributing valve back to its mid-position after the turbine gates have been moved, usually

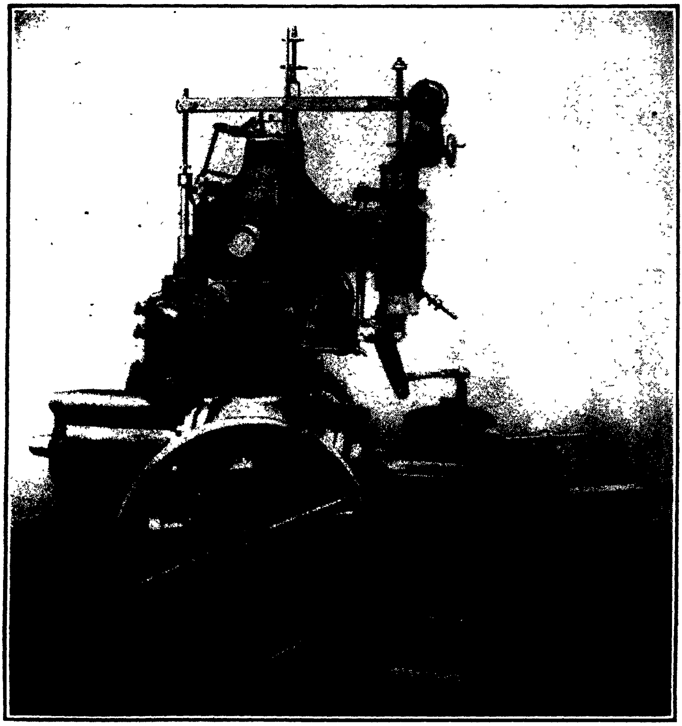


FIG. 419.—18,000 Ft.-lb. Capacity Oil-pressure Governor for Vertical Turbine. Regulating shaft extends up through base with operating lever keyed to it. Separate hand control through bevel gears on worm.

consists of light rods connecting the gate shaft or piston motion with the pilot valve through a floating lever or other device. Usually a dash-pot is also connected into the relay system; this materially assists in preventing overtravel and brings the unit back to normal without surging or hunting. The adjustment of the relay mechanism and dash-pot is very delicate, and the design of these elements determines, to a large extent, the sensitiveness and quality of the governor. Fig. 421 shows diagrammatically how this action is accomplished. In this diagram the flyballs (1) are connected to the floating lever (3). The distributing valve or regulating valve (6) is connected to one

end of the floating lever, and the restoring mechanism (7)–(8) is connected at the opposite end of the floating lever. The regulating cylinder or servomotor is shown at (9) and diagrams *A*, *B*, *C*, and *D* show the action of the various elements of the governor for " load off " and " load on " conditions.

The Oil-pressure System.—Fig. 422 shows a motor-driven oil pump with pressure tanks mounted above, such as is used with small governors. In this design the pump is of the gear type, located in the base of the tank, and is motor-driven, through a silent chain. The discharge from the pump is into the pressure tank, but a relief valve is provided so that when a certain pre-

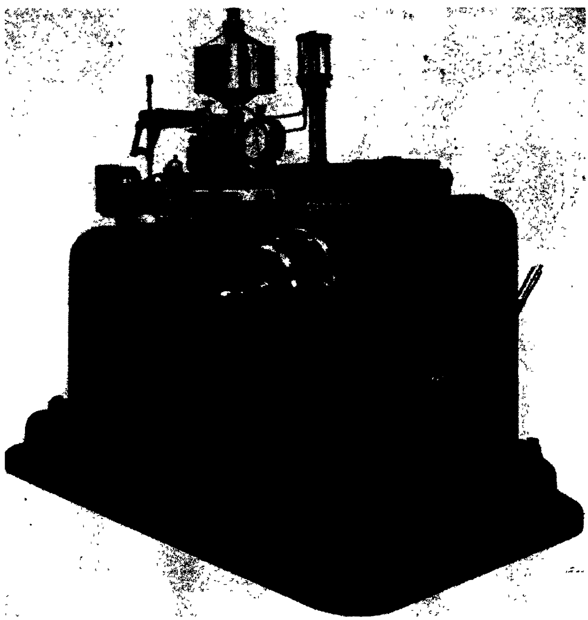


Fig. 420.—Control Stand for 100,000 Ft.-lb. Capacity Governor, Showing Gate Indicator, Tachometer, and Hand and Electric-operated Controls. (Pelton Waterwheel Co.)

determined pressure is reached in the pressure tank, the oil discharges back into the sump tank, which is located below the pump.

Some systems use water, with a small amount of soluble oil or potassium bichromate in solution, for pressure fluid, but this has not been found as satisfactory as oil, since these solutions tend to congeal and cause sticking of the pilot valves, and they cause greater wear on the pumps and other pressure parts. Practically any type of good-quality oil may be used with good success and without requiring replacement after several years' operation. Different manufacturers recommend different types of oil for their respective governors.

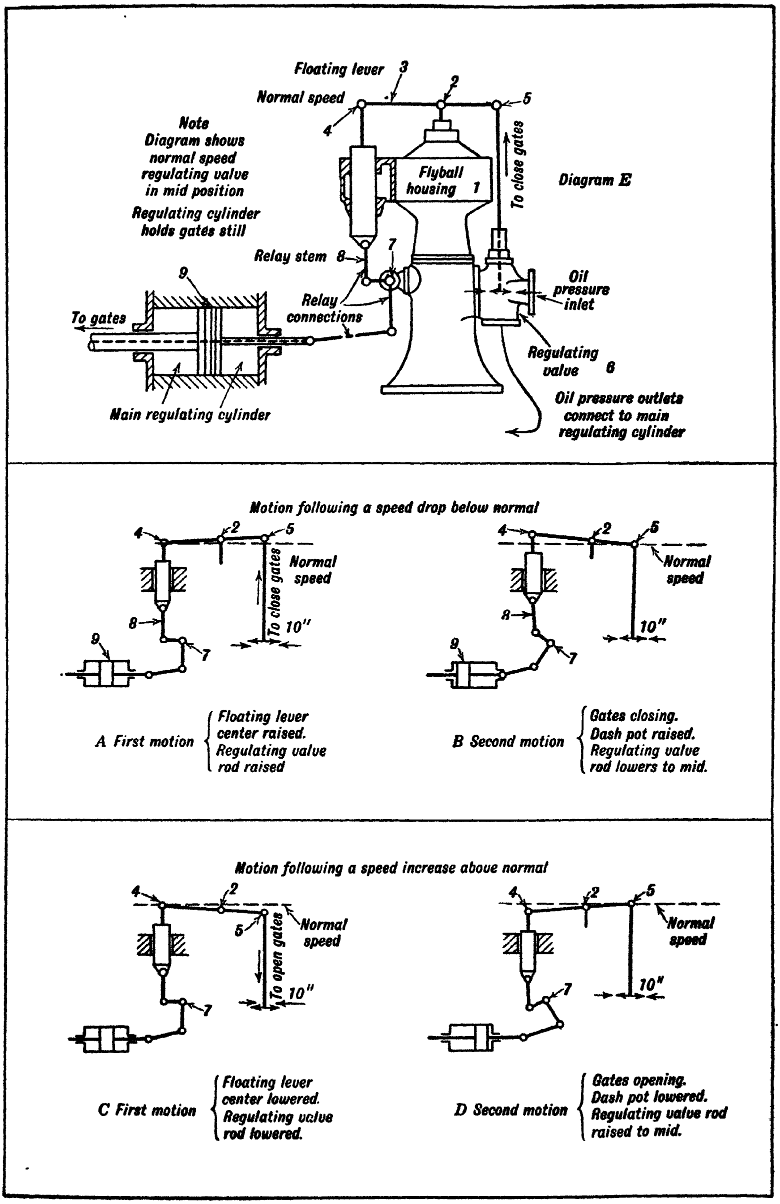


FIG. 421.—Diagrammatic Sketch of Governor, Showing Position of Cylinder, Dashpot, Floating Lever, Flyballs and Regulating Valve. Action of relays or restoring mechanism for "load on" conditions shown at *A* and *B*, action for "load off" shown at *C* and *D*.

Figure 423 shows a pumping unit for a large plant, consisting of two motor-driven gear-type oil pumps direct-connected to motors and also provided with an impulse-wheel drive, located between the pumps and connected by clutch couplings so that either or both pumps may be driven by the water wheel in case of failure of current supply to the motor drive. This type of pumping unit is suitable for a central oil system, where pressure for all the governors is obtained from one set of pumps. In this case one large pressure tank is usually located near the pumping system and a small booster tank is located near each governor so that, in case of rapid load fluctuations, there will be a sufficient supply of oil at hand to supply each governor until the oil in the long connecting pipes has had time to accelerate in order to supply the oil used.

Fig. 422. — Motor-driven Oil-pressure Pumping Unit for 30,000 Ft.-lb. Governor, Showing Location of Pressure and Receiving Tanks, Gear-type Oil Pump, Relief Valve, Gauge Glass and Pressure Gauge.

For large-sized turbines, the unit oil-pressure system is recommended, that is, each unit having its own complete pump and pressure system. Where there are a number of small or medium-sized units in the power house, the central oil system is frequently used, but this requires long lengths of large-sized tubing, and the failure of one of these tubes or of the main pump or tank, may shut down the entire power house, whereas with the unit system, only one unit will be affected, and if the systems are interconnected, this unit may be supplied with oil from one of the adjacent units. Oil is usually supplied under a pressure of 125 to 200 lb. per square inch. Threaded pipe fittings should not be used for oil under pressure. All fittings should be flanged and bolted together.

Fig. 423.—Motor and Impulse-wheel-driven Rotary Pumping Set for Large Central-governor Oil-pressure System with Clutch Couplings on both Sides of Impulse Wheel, so that it can be used for Driving Either Pump in Case of Failure of Electric Power Supply.

Usually the oil discharged from the regulating cylinder empties into a sump tank which is simply an open receptacle, usually placed below the floor line, and into which all of the oil may be drained. On interconnected systems there is usually only one large sump tank near the pumps, from which the pumps draw directly, although sometimes small sump tanks are located near each unit, these then draining into the main sump tank. The sump tanks should be located close to the pump suction and at such an elevation that

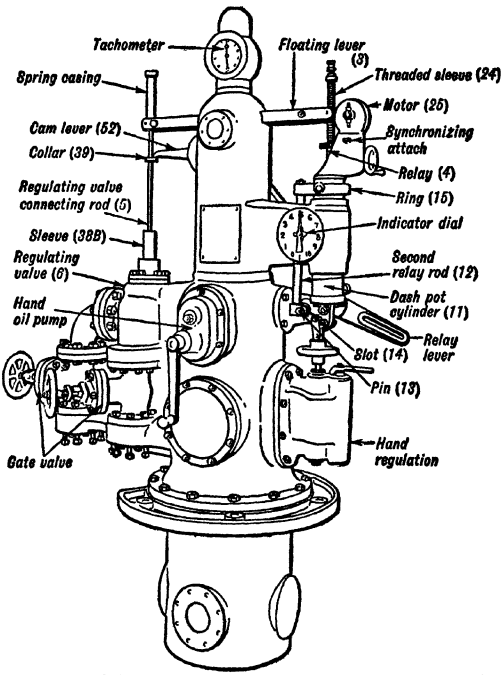


FIG. 424.—Governor Stand for 75,000 ft.-lb. Capacity Governor, Having Direct Connected Flyballs, Showing Location of Regulating Valve, Floating Lever, Tachometer, Synchronizing Motor, Dashpot, Gate Indicator, Hand-regulation Lever and Control and Hand-operated Gear-type Oil Pump for Emergency Use.

the suction head on the pump will be low, so that priming will not be necessary even after prolonged shut-down. Short lengths of pipe, both pressure and return, are desirable, as long lengths of pressure piping set up a time lag in the action of the governor, and long lengths of return pipe set up a back pressure on the cylinder which decreases the actual working force.

Small governors of 30,000 ft.-lb. capacity and less are usually equipped with an auxiliary hand-gear mechanism. The hand wheel located on the far side of the governor shown in Fig. 419 is for this purpose, and consists of a set of gears and levers connected to the gate shaft so that the gates may be moved

when pressure is not available. On larger units it is not practical to operate the gates by gears, but a small hand pump is provided so that oil may be pumped into the regulating cylinder, for moving the gates. Fig. 424 shows a governor stand which is used when the flyballs are mounted on the main shaft. The rod transmitting the flyball motion to the floating lever may enter overhead, or from below the floor line, passing up through the governor. The distributing valve, sometimes called the pilot or regulating valve, is located at the left, the dash-pot at the right, and the hand-operated gear pump is shown mounted in the center of the governor base. Below the dash-pot on the right-hand side of the governor, the hand-control valves are located. The lever below the dash-pot at the right has two positions, one for governor control and one for hand control. When set for governor control, as when operating under control of the flyballs, the gates may be moved with pressure from the oil-pressure system by turning the small hand wheel above the dash-pot, the position of the gates being indicated by the dial at the side of the dash-pot. If the lever is set on the hand-control position, the motion of the flyballs does not control the unit, but the gates may be moved by turning the hand wheel below the dash-pot; and if pressure is not available in the pressure system, sufficient pressure may be obtained by turning the hand pump mounted on the governor, until the gates have reached whatever position is desired.

In large hydro-electric systems when there is more than one unit in the system, the individual governors on the units must be adjusted so that the units operate properly in parallel. If it is desired that all the units in the system divide the load changes proportionately, then all the governors must be set to the same percentage of regulation. By percentage of regulation is meant the speed change between no-load and full-load. This varies from $\frac{1}{2}$ of 1 per cent to 2 or more per cent, although 1 to 2 per cent is the average for hydro-electric systems. In addition to this adjustment, the flyballs must be sensitive so that they respond readily to speed changes.

Size of Governor Required.—The capacity of the governor is measured in foot-pounds and is the product of the piston area in inches, the pressure in pounds per square inch, the stroke of the piston in feet, and the number of pistons, usually one or two. The capacity of governor required may be calculated approximately from the following formula:

$$C = \frac{50P}{H^{1/4}}, \qquad (171)$$

where C = capacity of governor required, in foot-pounds;
P = horse power of unit;
H = head, in feet.

This is approximately correct for medium-sized spiral-cased units, although for large-sized high-head units this may be decreased from 10 to 20 per cent, and for low-head open-flume units and horizontal open-flume units, these values should increase from 10 to 20 per cent. It is desirable that the oil

pressure should not exceed 150 lb. per square inch, although some governors are operating satisfactorily at pressures above 200 lb. per square inch.

334. Speed Regulation.—By speed regulation is meant the percentage change in the speed of the unit for a given change in load. This percentage of speed change varies directly as the governor time and as the horse-power change, and inversely as the flywheel effect of the unit and system which takes the load change. It is also affected by the penstock conditions of the unit. For an open-flume turbine the speed change in per cent may be calculated from the following formula, for both speed rise and speed drop:

$$I = \frac{80{,}000{,}000P't}{R^2W}, \quad \ldots \ldots \ldots \quad (127)$$

I = per cent speed variation expressed as a whole number;
P' = horse-power change on unit;
t = governor time in seconds between instant of load change P' and time of completion of gate readjustment;
R = normal speed of unit, revolutions per minute;
W = flywheel effect of unit, usually called WR_2—Product of rotating weight and radius of gyration in feet.

The above formula, which gives the theoretical fluctuation in speed, is subject to several corrections. The principal one for speed rise is based on the fact that most water-wheel runners, under no-load conditions and normal head conditions, will not exceed 80 per cent above their normal speed under any condition of load change.

This correction may be applied approximately from the following formula, all percentages being expressed as whole numbers:

$$I' = \frac{I}{100 + \dfrac{100I}{r - 100}}, \quad \ldots \ldots \ldots \quad (173)$$

where I' = per cent speed rise, corrected for runaway characteristics of runner;
I = uncorrected per cent speed rise, computed from Eq. (172);
r = runaway speed of unit in per cent of normal speed as given in Fig. 393.

Given a computed uncorrected speed rise of 40 per cent with a runner having a runaway characteristic of 180 per cent, the corrected speed rise will be only 26.7 per cent, but this will not affect the speed drop, which is not affected by the runaway characteristic.

The above formula may be used for open-flume installations, either vertical or horizontal, and where the pipe line or penstock is relatively short. Curves in Fig. 425 show the speed rise for "load-off" conditions, for different governor times and for different regulating constants. These curves are corrected for runaway-speed characteristics of 180 per cent, and the part-load speed rises are computed, assuming a governor lag of ¼ second: Run-

away-speed characteristics of 170 or 200 per cent will not materially affect these values. The regulating constant C, used for these curves, is derived from the following equation:

$$C = \frac{R^2 \cdot W}{P}, \qquad (174)$$

where C = regulation constant;
R = normal speed of unit, revolutions per minute;
W = flywheel effect of unit, WR^2;
P = horse power of unit.

Substituting in Eq. (172), the equation for speed change becomes:

$$I = \frac{80{,}000{,}000t}{C}, \qquad (175)$$

where I = per cent speed variation;
t = governor time in seconds between instant of load change and time of completion of gate readjustment;
C = regulation constant as given in Eq. (174).

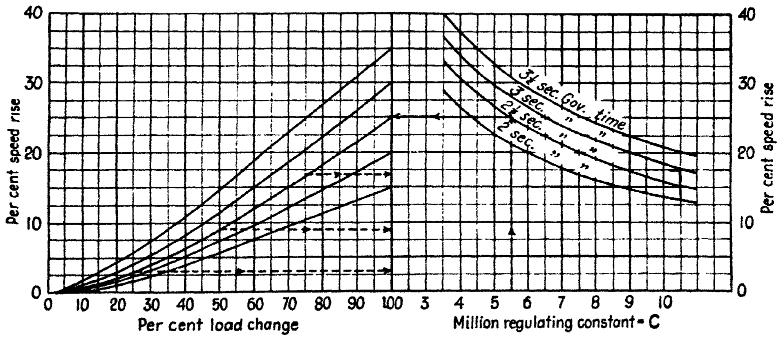


FIG. 425.—Speed Variation for "Load Off" Curves Showing Relation between Per Cent Speed Rise for Various Governor Times and Regulating Constants Corrected for 180 Per Cent Runaway Based on ½ Sec. Gov. Lag.

Given a regulating constant of 4,000,000 and a governor time of 2½ seconds, following over to the left we find that the speed rise for 100 per cent load-off is 30.7 per cent, and the speed rise for fractional load changes may be found by following down the curves at the left. These speed changes are computed on the assumption that the unit is operating on a water rheostat load, so that only the WR^2 of the unit itself is effective.

This regulating constant is a factor which is used a great deal in discussing speed regulation values, from 4 to 5 million usually being used for small units of the open-flume type, and values from 6 to 7 million for larger units of the spiral-casing type, values above 6 million sometimes being required for plants having long penstocks.

Figure 426 shows the speed drop resulting from "load-on" conditions

plotted on the same basis as these curves in Fig. 425 except that there is not correction for runaway speed.

The effect of penstock conditions on speed regulation is very important, any change in the velocity of the water in the pipe causing a change in the pressure. When "load-off" conditions occur and the governor causes the gates to close, this decreases the velocity in the pipe, which causes a pressure rise; or it increases the effective head, acting on the turbine, which further tends to increase the speed rise; so that for plants having long penstocks, the

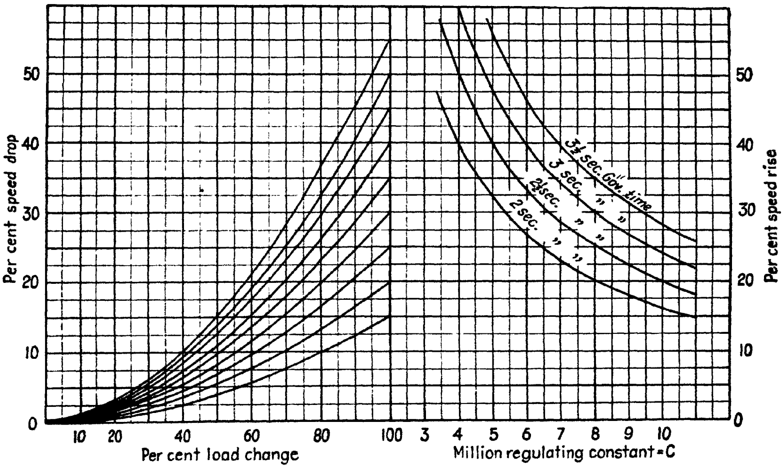


FIG. 426.—Speed Variation for "Load On" Curves Showing Relation between Per Cent Load Change and Per Cent Speed Drop for Various Governor Times and Regulating Constants Based on ½ Sec. Gov. Lag.

values of speed rise obtained from Fig. 425 must be corrected by multiplying by the three-halves power of the pressure rise, or

$$I'' = I'(1 + J)^{3/2}, \quad \ldots \ldots \ldots \quad (176)$$

where I'' = per cent speed rise corrected for both runaway speed and pressure rise;

I' = per cent speed rise corrected only for runaway speed;

J = ratio of increase in head to normal head, called pressure rise.

For conditions of "load-on" with pressure drop, the values of speed drop obtained from Equation 172 must be divided by the three-halves power of the pressure drop.

$$I'' = \frac{I}{(1 - J')^{3/2}}, \quad \ldots \ldots \ldots \quad (177)$$

where I'' = per cent speed drop corrected for pressure drop;

I = per cent speed drop uncorrected for pressure drop;

J' = ratio of decrease in head to normal head, called pressure drop.

The amount of pressure rise may be computed approximately from the formula:

$$J \text{ or } J' = \frac{LV}{GtH}, \quad \ldots\ldots\ldots \quad (178)$$

where J or J' = ratio of change in head to normal head;
L = penstock or tunnel length, in feet;
V = velocity change, in feet per second;
G = acceleration due to gravity, 32.2;
t = time, in seconds, in which velocity change, V occurs;
H = normal head on unit, in feet.

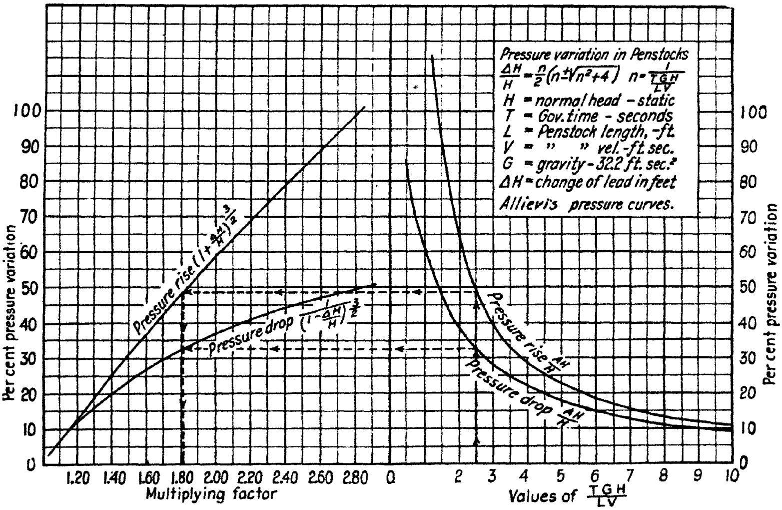


FIG. 427.—Multiplying Factors for Corrections Figured Speed Variation for Penstock Pressure Variation.

The values of pressure rise obtained by this formula are not quite correct, because the elasticity of the pipe line and the compressibility of the water affect the pressure rise somewhat. A formula developed by Mr. Allievi is plotted in Fig. 427, the value, $\frac{TGH}{LV}$, being plotted horizontally and pressure rise vertically. On the left-hand side of this figure, the multiplying factors are shown. This factor is to be used to multiply the speed-rise figures obtained from Figs. 425 and 426 to obtain the correct pressure-rise or pressure-drop values. The above method may be used for figuring the approximate speed and pressure rise under the ordinary conditions, and although not exactly correct, gives a fairly close value for practical use.

The proper coordination of penstock diameter, WR^2, and governor time, should be looked into carefully in the design of any plant, as it frequently

works out that there is a most economical combination which will result in a considerable saving. The maximum speed change allowable, however, depends largely on the characteristics of the system to which the unit is to be connected, and the amount of WR^2 which will be tied into the system. Where it is desirable to obtain close speed regulation, it is frequently necessary to install either a surge tank or a pressure regulator, or both.

Pressure regulators may be used either for " water-saving " or " water-wasting " operation. Their operation is fully described in Sec. 337. Where they are operated for " water-saving," the pressure regulator is closed during normal operation of the turbine; but when load is rejected and the governor closes the gates of the turbine, the pressure regulator opens simultaneously so that there is practically no pressure or velocity change in the pipe line. The pressure regulator will then close slowly, so that the velocity changes will not set up serious pressure rises. Pressure regulators operating under these conditions, however, do not benefit the " load-on " conditions, so that the governors must be adjusted to suit the penstock conditions for " load-on." If the pressure regulator is set for " water-wasting," it is so designed that under all load conditions the velocity in the pipe line remains constant. If load is rejected from the turbine and the gates closed slightly, the pressure regulator opens up a sufficient amount to maintain the velocity constant. If the load increases and the gates open, the pressure regulator closes an equivalent amount; but where the supply of water is limited, this does not allow efficient use of the water.

Where it is possible to install a surge tank near the power house, this provision is a very reliable source of protection against both pressure rise and pressure drop, as a surge tank is beneficial both for " load-on " and " load-off " conditions. Its area should be sufficient to supply the water necessary to run the unit when " load-on " conditions occur until the velocity in the upper part of the pipe line has increased sufficiently to take care of the added demand. " Load-off " conditions may be taken care of by allowing the surge tank to spill over, although in cold climates trouble may be caused by the freezing of this spilled water.

Under all conditions it is desirable that the surge tank be located as close as possible to the power house, in order that pressure variations caused by velocity changes in the penstock between the power house and the surge tank may not be appreciable. The matter of surge tanks has received a great deal of attention and there are many articles written on this subject, particularly on the Johnson differential surge tank which has many advantages for certain conditions. In cold climates, it is usually necessary to provide for heating the surge tank during winter, as ice will render it useless and endanger the pipe line. This is another reason why the surge tank should be located as close to the power house as possible, in order that the same heating system may be used for both.

Bursting plates have not been found to give adequate protection against pressure variation. They are usually designed to burst at a certain predetermined over-pressure and thus limit the pressure rise in the pipe, but the resulting pressure drop after the bursting plate goes out may cause more

serious damage, even collapsing the pipe. Bursting plates have also been found unreliable as to uniform bursting pressure, and for this reason their use has been practically discontinued. When it is possible to design a large number of bursting plates to rupture at slight increments of over-pressure, this disadvantage may be overcome, but this has not yet been successfully worked out.

It is not necessary to allow excess turbine capacity above the rated horse power of the unit for purposes of governing. The maximum or rated capacity of present-day hydraulic turbines is fixed by the maximum gate opening; and for this the gates are blocked at a point on the power-efficiency curve where an increase in the power could still be obtained with greater gate opening, and where the efficiency curve is still sloping down gradually but not steeply. If the blocking were removed and the gates of a turbine opened indefinitely, a point called "overgate" would be reached where the power output of the turbine would actually decrease with greater gate opening. This point of "overgate" is usually about 10 to 25 per cent greater than the full gate opening for which the gates have been blocked. Therefore, by altering the point at which the gates are blocked, practically all units may be made to give greater capacity than rated, if greater capacity is actually needed for governing purposes.

However, due to the modern practice of interconnection of power systems, sudden, large changes in load do not occur and governing is not a serious problem.

335. Plants without Governors.—Recently there have been placed in operation several plants not equipped with governors, where the only control of the units was by hand mechanism or by a motor. The first of these plants was equipped in this manner because of the extreme penstock conditions and the desire to save the additional cost of the surge chamber or pressure regulators. Since these plants were to operate in parallel with other units equipped with governors, they could be set for a constant output; and their load was varied only to maintain a margin for regulating on the governor-equipped plants or to suit the flow.

The most essential feature in a unit of this kind is that all of the rotating parts must be designed to stand prolonged runaway under the maximum head conditions possible with the gates wide open. While this condition may only occur at rare intervals, such as when there is line trouble or when for some other reason the entire load is lost, it is sure to occur at some time, and the units should be designed accordingly. Some of these plants, which have motor-operated mechanisms, are equipped with an over-speed flyball control switch which starts the motor in the closing direction when the speed reaches approximately 10 per cent or some other set value above normal. The gate mechanism is designed to close in about 50 seconds.

There is a feeling among some engineers that in the future fewer and fewer of the hydro-electric plants will be equipped with governors. The reason for this prediction is as follows: As electric power systems become larger, the relative load changes become less so that in a large system serving a variety of loads it is possible that one or two units may take the entire fluctuation of the

system. In such a system a load dispatcher would watch carefully and adjust the loads on the blocked units so that the units doing the regulation would always have available a certain margin for either increase or decrease in demand.

Existing large systems at the present time usually operate with many of the units blocked or on the load-limiting device, so that the governors do not become active unless there is a large decrease in load. In other words, a great many of the present-day governors are merely safety devices and as such could be constructed on much simpler and more economical lines. A large percentage of the work of the operating forces is taking care of and inspecting governor equipment, such as pumps, belts, motors, relief valves, air compressors, etc. A large percentage of the shut-downs in existing hydro-electric systems are caused by some trouble in the governor equipment, the majority of which will be eliminated with the simplified type of governor. Many large systems are operated in parallel with steam turbines, and it is suggested that the steam turbines might well take the load fluctuations, the hydro-electric units carrying the base or constant load.

336. Automatic Control Systems.—In recent years, on account of the high cost of labor, there is a tendency toward automatic equipment. Such equipment greatly reduces the number of operators required and makes it possible to develop small hydro-electric sites which could not warrant their existence if the salaries of a crew of operators were to be maintained. While automatic equipment may still be considered in its infancy there are three principal types.

Full Automatic Control.—A unit designed for full automatic control is usually located at a considerable distance from the point of power consumption and the method of operation is approximately as follows: As long as the transmission line to which this unit is connected is energized, the unit will operate. When the switches connecting this transmission line to the system are thrown in, the first operation usually is to start up the governor pump which builds up oil pressure and slowly opens the gates of the turbine. As the unit comes up to speed, flyball-controlled switches connect the generator to the line at approximately synchronous speed through amortisseur or damper windings which limit the flow of current through the generator until it has been pulled into phase. The gates of the turbine then continue to open up until that point in the gate opening has been reached which corresponds to the speed of the system and the load setting of the governor. The unit then continues to operate until it is shut down by disconnecting the line switches at the other end of the line; or, it may be shut down by some of the protective devices on the unit itself.

Protective devices may include some or all of the following: bearing temperature relays, generator-winding temperature relays, water-pressure relays, overload relays, reverse-current relays, low-voltage relays, ground relays, and relays connected to any part of a hydro-electric unit for which it is desired to provide protection. In case the unit has been shut down by some protective device of this kind, it cannot again be started up until a repair man or operator has corrected the trouble. Units so equipped are usually inspected once each

day, at which time they are greased and the lubricating systems are gone over.

Remote-control Stations.—Remote-control stations are largely the outgrowth of large units which are controlled from the switchboard rather than from the operating floor, since the operator at the switchboard is in a better position to watch the operation of the unit, by observing the action of the various load meters, than is the turbine operator on the turbine floor. Operation from the switchboard usually consists of controls to the synchronizing and load-limiting device on the governor, the switch for the governor oil pump, the controls for operating the electrically operated switches which connect the generator on to the transmission line, as well as contact switches for shutting the unit down quickly in an emergency. These same electrical connections may be run to a distant station, where the operator in control, observing the action of the unit through the frequency meters and load meters, can control its operation. These various controls are usually handled through separate wires for each, although a single pair of wires and a selector mechanism, such as is used in automatic telephone work, can be employed. Frequently this remote-control system is used in combination with an automatic-control system, the unit being started up by the remote-control system and the load adjusted to suit demands or according to water supply, the automatic equipment being provided to take care of any emergency shut-down and other features.

Float Control.—The third type of automatic equipment is designed to operate the unit so as to utilize most effectively the full flow of the stream. Two or more float-operated contact switches are provided, either in the forebay or connected to the penstock, and actuated by pressure. When the water in the forebay rises to a predetermined height, the unit is started up, synchronized, and placed on the line, carrying its full rated load until the water in the pond reaches a predetermined low level, at which time the unit is shut down tight. Combinations of this system may be arranged by adjusting the full load which the unit may carry to suit approximately the average flow for the existing season; or five or ten different level-operated switches may be provided, so that when the water is at the maximum height the unit carries full load, when it drops 1 ft. to the next switch the unit carries .9 load, and when the water has dropped 10 ft. the unit is shut down entirely. With this condition the unit would operate practically continuously and would carry a load corresponding approximately to the stream flow.

With this system, the protective devices, similar to those of the remote-controlled and automatic-controlled stations, are used. This system is also used in connection with both the automatic and remote-controlled systems to regulate the amount of load which the unit may take.

337. Pressure Regulators.—Pressure regulators are devices, usually used with hydraulic turbines where long penstocks exist, to prevent rapid velocity changes in the pipe line. Pressure regulators are usually designed to open when the turbine gates close. They may be so designed that they either close slowly after steady conditions have been maintained, or they may remain open, maintaining a constant velocity in the pipe line, and will be closed only

when the turbine gates are again opened. The governor-controlled type of pressure regulator, one form of which is shown in Fig. 428, consists of a valve connected to a hydraulic cylinder, and is normally held closed while the turbine is in operation. The usual method of controlling the pressure-regulator opening is to have a connection from the shifting ring of the turbine to a pilot or control valve which allows the water to flow out from under the piston of the pressure regulator, allowing it to open. Some pressure regulators, however, are connected rigidly to the governor mechanism as shown in Fig. 429, so that the force exerted by the governor causes the pressure regulator to open. These are called governor-operated.

The valve of the pressure regulator is usually of the mushroom type and for high-head conditions should be provided with a renewable seat on the valve and a renewable seat where the valve rests. Usually the cylinder is operated with penstock pressure and it should be bronze-lined to prevent rusting. A filter system should be provided so that foreign matter does not clog the pilot valve of the pressure regulator.

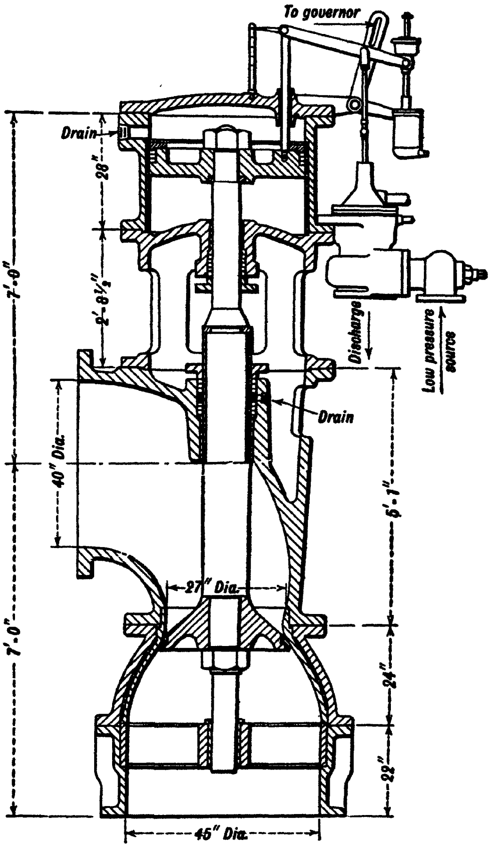


FIG. 428.—Governor Controlled Pressure Regulator for Water-saving or Water-wasting Operation. Pilot valve at right is operated from gate mechanism and when gates close releases pressure from within cylinder which allows mushroom valve to open to maintain constant velocity in penstock. Dashpot then controls pressure regulator closing time or if locked pressure regulator remains open acting water wasting. (Allis-Chalmers Mfg. Co.)

A water-wasting pressure regulator is designed to maintain constant velocity in the pipe line at all conditions of load. The opening of the pressure regulator is adjusted in proportion to the gate opening so that water not used by the turbine will be discharged through the pressure regulator. This system does not allow economical use of the water, but it may be necessary where there are rapid load fluctuations, especially where there may be an

instantaneous demand for a large increase in power. Water-saving pressure regulators are designed to open only when the turbine gates are closed rapidly, and are designed to close slowly so that there will be no serious over-pressure set up in the penstock. The closing time of the pressure regulator may be set to suit local conditions. Many pressure regulators are now constructed so that they may be operated either as water-saving or water-wasting.

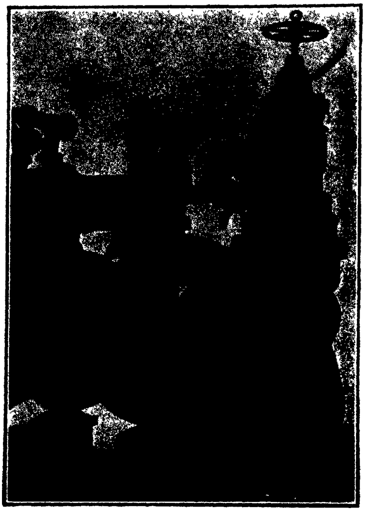


FIG. 429.—Governor-operated Pressure Regulator. Power from servo motor opens relief valve when turbine gates close. Water pressure then causes valve to close slowly for water saving. (Pelton Waterwheel Co.)

338. Valves.—Valves of the butterfly, Johnson, or gate type are used at the inlet of the spiral casing of many medium and high-head units. They form an integral part of the hydraulic-turbine equipment, and are usually purchased with it. The safe and reliable functioning of these valves is an important item in the operation of a hydraulic plant, and their design and construction should be given as great care as other parts of the hydraulic equipment, if not greater. The various types of valve and the details of their construction and methods of operation are fully discussed in Chapter XVII of this book.

339. Spare Parts. — Some spare parts are recommended on practically every type of hydraulic-turbine equipment. Where there is danger of breaking the runner with foreign material, a spare runner is desirable, if not too expensive, especially on small units. From two to six spare guide vanes are desirable, and at least a dozen guide-vane links, if they are of the breaking type. With lignum-vitae water-lubricated bearings, it is desirable to carry an additional bearing shoe, already lined with lignum-vitae and bored to size, but this should be kept in moist sawdust to prevent drying out. It is usually not considered necessary to carry a spare babbited bearing as there is usually very little trouble with these, although with the ring-oiled horizontal bearing, an extra bearing shell is sometimes carried in stock. For the governor equipment on large units, a spare set of pump gears and a regulating valve are desirable; but on small units scarcely any spare parts need be carried for the governor, as the unit can be run on the hand control in case of governor trouble.

340. Miscellaneous Auxiliary Equipment.—Other items of equipment sometimes required in a power house in connection with hydraulic equipment include headgates, penstocks, drain valves and piping, ejectors, air compressors, wrenches and tools, inspection man-holes, pumps for pumping out submerged parts below tail-water level, and an adequate means of supplying

continuously, winter and summer, such cooling and lubricating water as is required for cooling governor oil and oil-lubricated bearings, and for lubricating lignum-vitae water-lubricated steady bearings.

The matter of headgates and penstocks is discussed in Chapters XVI and XVII. Headgates are an important feature. They must be capable of being closed in an emergency in a reasonably short time. Penstocks must be strong enough to withstand the maximum water hammer imposed by the gate movement, and must be designed in coordination with the generator WR^2 and governor time so that the most economical combination of water velocity (which means penstock diameter, and therefore, penstock cost) WR^2, and governor time is obtained.

Drain valves of some sort are required in all water-power plants. Usually simple flap traps are sufficient for open-flume and low-head concrete spiral-cased units. Plate-steel and cast-metal cased units, as well as many concrete spiral-cased units, require drains equipped with gate valves, frequently with the operating mechanism led up to the operating floor. Such valves must be of liberal diameter to carry away the maximum headgate or valve leakage, and must be of rugged design so that they can be opened with full head on the valve and closed again, if necessary, with full spouting velocity through the valve. They must also have a rugged operating mechanism which will not twist off in case some obstruction clogs the valve.

Ejectors are used in some low-head open-flume and concrete spiral-cased settings known as "siphon settings," and are used to eject air from that part of the enclosed chamber which is above head-water. They must be properly screened to prevent clogging, and the suction pipe to the top of the chamber, as well as the ejector itself, should be protected so as not to be damaged by "dead heads."

Air compressors are a desirable accessory in medium and large plants to provide air pressure for the generator brakes for stopping the unit, as well as for replacing the air cushion on governor pressure tanks. In small governors a small amount of air can be bled through the oil pump, but this is to slow when it is desired to get a unit into service quickly, and it may also cause objectionable noise in the pump. Air pressure is also very useful around a power house for blowing out generators, running small drills and jack hammers, and for small repair work. Usually only a 15 or 20 cu. ft., 175-lb. pressure compressor is required.

A complete set of open-end wrenches, as well as any special box wrenches and sockets required for special work on the machinery, should be available and should be mounted near the units so as to be readily accessible. In addition, a number of jacks, bars, drills and jack hammers should be available around a large power house, although these may not be required for smaller stations.

Inspection man-holes, especially in concrete draft tubes and concrete spiral casings, are not usually furnished with the turbine but form an essential part of the installation. Their design should be such that they are rugged, but not so cumbersome as to make removal for inspection too difficult.

Some plants, where the tail-water rises above the bottom of the turbine

during a part of the year, are provided with gates for the draft-tube outlet and pumps for pumping out the draft tube and casing so that inspection and repairs can be made during high tail-water periods if necessary. Usually one pumping unit is sufficient, connections and valves being provided so that any draft tube may be pumped out at will.

Power-house cranes are now considered a necessity in all except very small power houses. Their design is important and is discussed fully in Chapter XXVII.

The supply of a continuous and adequate amount of water for cooling oil bearings, or lubricating lignum-vitae bearings, is important. The customary method of supply is to tap the penstock or casing and filter the water through a twin strainer, either half of which can be cleaned without interfering with the operation of the other half. This is ordinarily considered satisfactory; but in cold climates where frazil ice forms, or where chips or leaves are troublesome at certain times of the year, such strainers have been known to clog up in a few minutes' time, and a number of lignum-vitae bearings have been badly burned because of this. Revolving screens, heater tanks, and special intakes built into the dam, have been used at various plants to remedy this trouble.

341. Hydraulic-turbine Tests.—Tests of hydro-electric plants are desirable, as they furnish a reliable means of knowing just what the output and efficiency of the units are. They are desirable with new equipment, to make sure that the manufacturer has met his performance guarantees. They are desirable with old units, to ascertain whether or not the units have decreased in efficiency owing to wear, pitting, or some other cause; and they are also desirable with old units in order to know their operating efficiency, as it may so happen that the increased efficiency which could be gained by replacing or rebuilding the old turbines would more than pay for the cost of the work.

Measurement of Power.—The power output of a hydro-electric unit may be quite readily obtained by calibrated wattmeter readings; but individual current transformers and potential transformers should be used with test wattmeters, and these should all be calibrated, together with the same leads and at the same power factor and voltage at which the test will be run. A competent electrical engineer should be in charge of the connections and handling of the instruments. On some recent tests of large units it was found that the error in the electrical readings, of 2 per cent, was much greater than the error in the water readings.

For the measurement of head, horse power, and quantity, the Machinery Builders' Testing Code, adopted in 1917, is the best record. The American Society of Mechanical Engineers has also recently issued a testing code for hydro-electric plants, which may be obtained from the secretary.

Measurement of Head.—The head on open-flume turbines is usually taken as the difference in elevation between the water in the flume near the turbine and the tail-water just outside of the power house. Correction must be made for the velocity head necessary to discharge the amount of water used out of an area equal to the downstream end of the draft tube.

For turbines with enclosed casings of the spiral type, the head is usually

taken as the difference between the tail-water elevation, corrected for discharge velocity head, and the pressure at the inlet end of the spiral casing, this inlet pressure being corrected for the velocity head at that point. Losses in penstock and racks are not taken as part of the turbine loss. Similarly, in cylindrical-cased units, the velocity of the inlet is not charged against the turbine, because the cylindrical casing does not use this velocity efficiently.

Quantity of Water.—There are several methods of measuring quantity of water. The most popular ones at the present time are the Gibson method, using the pressure variation in the pipe line caused by a change in flow; and the Allen salt-velocity method, measuring the time required for a dose of salt to pass between two stations, the salt being detected by the variation in an electric current passed through the water. Other methods include the Pitot-tube method, in which a calibrated Pitot tube is used to traverse a penstock, traverses being made at two or three different angles, but in the same plane. Calibrated current meters are used with open-flume and low-head units to determine the average velocity in a given area, one reading being taken for approximately every 9 sq. ft. of area. This brings the meter points approximately 3 ft. apart. These are used extensively for open-flume and open-canal measurements, but care should be taken in the selection of the current meters. Some current meters over-register for disturbed flow, this error amounting to as much as 20 per cent in some conditions. Some current meters under-register in disturbed flow, and in all cases the instruments should be in the hands of experienced persons.

Some plants are constructed with a weir in the tail race, but some means must be provided to quiet the water, preferably with baffles to still any disturbances. A weir practically loses 2 to 3 ft. effective head for the plant; it is therefore a continual bill of expense, in that it reduces the capacity of the plant. Venturi meters are installed in pipe lines of high-head turbines, but when so installed they cause a friction loss of from 1 to 2 ft. head, which is not to be recommended. However, it will be found that practically every spiral-cased turbine has a contraction at the inlet of the casing, and this contraction may be carefully calibrated by Pitot tube or other water measurements and a permanent gage established, so that it may be used exactly as the Venturi meter is used with no added loss in head.

This method of utilizing the contracting part of the casing inlet for a Venturi meter has been described in a paper read before the May, 1925, meeting of the American Society of Mechanical Engineers by Mr. E. A. Dow, entitled, "Mechanical Features Affecting the Reliable and Economical Operation of Hydro-electric Plants."

It is not the intention here to give a comprehensive treatise on testing, as it is very difficult to cover the many points which may come up during a test. Common sense is the first requirement, and an experienced man will save much time and delay in the conduct of such a test. Practically all of the manufacturers of hydraulic turbines have engineers available who have had experience in various methods of testing, and they are always willing to give counsel and assistance.

There are also several consulting engineers who will undertake to conduct power-house tests and who usually have available special instruments for this purpose. The majority of the larger power companies are also usually equipped with calibrated electrical instruments; and recently many of them have acquired instruments for making water measurements, as they find that a frequent check on the water consumption is very desirable.

Wherever there is a penstock or canal of uniform cross-section leading up to the turbine, the salt-velocity test may be readily made. Salt solution is injected into the upper end of the penstock with pressure, and the time of injection is taken as the starting time. Electrodes stretched across the penstock, some distance below the point of injection, are connected to a recording or curve-drawing ammeter, and when the dose of salt passes the electrodes a peak occurs on the curve. The amount of equipment required for such a test is not excessive and it may be installed and the tests completed in a few days.

With the Gibson method of water measurements, the pressure-recording apparatus is connected to the penstock near the turbine. After readings of power and head have been taken at the output at which the efficiency is desired, the turbine gates are caused to close slowly, this causes a pressure wave which is recorded by the apparatus. From these data the quantity of water flowing during the time previous to the moving of the gates can be computed. To use this method, it is necessary to have an engineer experienced in the conduct of such tests, in the operation of the recording apparatus, and in the computation of the results.

342. Guide for Purchasers of Hydraulic Equipment.—When asking for quotations on hydraulic turbines and accessories, the engineer should give the manufacturer complete information regarding the physical characteristics of the site and the type of unit desired.

The physical characteristics are:

1. Maximum, minimum, and average static head.
2. Fluctuations in head and tail-water elevations.
3. Length and diameter of pipe line, if any, and preferably a profile of same.
4. The net head for which the turbines are to have the best efficiency.
5. The shaft horse power desired at a given net head.
6. The speed at which the turbines are to operate.

The question of suitable and proper speed for the units should be definitely settled with the various water-wheel manufacturers, by asking them, some time before calling for final prices, what speed they recommend for the given conditions. It may be desirable to ask for both turbine and generator bids on two different speeds, so as to compare the cost of the units, together with their relative merits as to efficiency and the probability of pitting at the higher speed.

At the time of asking water-wheel builders for their recommendations as to speeds, it is well to obtain their advice as to WR^2 desirable in the generator, allowable penstock velocities, and spacing of units in the power house. With this information at hand the prospective purchaser can issue a much more

practical and more complete call for tenders, and this preliminary discussion may save both him and the tenderers much time and expense.

The unit desired is usually described in a preliminary specification or call for tenders which gives in detail the type and character of turbine and accessories. In all cases it is desirable to consult one or more of the manufacturers before deciding on the type of unit, as their suggestions may save a considerable amount of time in arranging the details of the installation.

The specifications should include:

(1) The type of wheel installation.
(2) The per cent of full gate power at which it is desired to have the point of best efficiency.
(3) Whether the most efficient type of unit is desired or whether efficiency is not of vital importance.
(4) Whether a reliable unit or a chaper, less reliable one is preferred.
(5) List of accessory equipment to be included, such as penstock valves, relief valves, steel draft tubes, governors, etc.
(6) Imaginary limits to the turbine equipment within which the manufacturer is to furnish *all* necessary parts.
(7) Location of governor equipment.
(8) Whether structural steel supports are to be included. These are usually furnished by purchaser.

The foregoing comments are in no way complete, and suggestions for additional information which should be included in the preliminary specifications are given in a " Form for Tabulation of Turbine Quotations," included hereinafter.

It is very difficult to obtain quotations which are strictly comparable unless complete preliminary specifications are provided. To purchase from the lowest bidder without careful comparison of the specifications, accessories, details, and other influencing characteristics of the different quotations is the height of folly. Unless the preliminary specifications are sufficiently clear to make it certain that all bidders are quoting on the same class of apparatus, many conferences, and revised quotations and much study will be necessary to get all bids lined up for comparison.

To assist the engineer in determining whether the manufacturers have included in the bids all the requirements of the specifications, and for a comparison of the bids, the following form is given.

Form of Tabulation of Turbine Quotations

General

1. Job.
2. Manufacturer.
3. Number of units.
4. Setting.
5. Casing.
6. Power of each unit.
7. Head.
8. At turbine rating (6).

9. Average effective.
10. Maximum (without water hammer).
11. Minimum.
12. Direction of rotation.
13. Dates of delivery.
14. Parts for concrete.
15. Complete units.
16. Drawings of overall dimensions.
17. Complete drawings.
18. Dimensions of unit affecting size of substructure.
19. Shop inspection.
20. Shop assembly.
21. Length of guarantees.
22. Shop painting.
23. Is field erection included in bid?
24. Daily rate of construction superintendent if (23) not included.
25. Diameter of runner.
26. Speed of units at normal operation and at runaway speed.

Weights and Price

27. Shipping weight.
28. Heaviest piece.
29. Total apparatus.
30. Weight of runner, shaft, and hydraulic thrust.
31. Total price and terms of payment.
32. F.O.B. where?

Efficiencies

33. Guaranteed efficiencies.
34. Full load.
35. ....% load.
36. ....% load.
37. Average between ....% and ....% load.
38. Tested efficiencies.
39. Full load.
40. ....% load.
41. ....% load.
42. Average between ....% and ....% load.
43. Where tested?

Details

44. Length and size of penstock reducers.
45. Wheel casing.
46. Type.
47. Material.
48. Thickness.
49. Drain pipes, fittings and valves.
50. Manholes and material of bolts.
51. Draft-tube manhole and cover.
52. Turbine pit-liner material and length above center line of turbine.
53. Speed-ring material.
54. Turbine gates.
55. Material.
56. Balanced at what gate?
57. Inside or outside operation?
58. Bushings or babbits?
59. Stuffing boxes.
60. Method of lubrication.
61. Removable wearing surfaces.

62. Top of gate.
63. Bottom of gate.
64. Turbine-gate rigging.
65. Shifting-ring bearing linings.
66. Links designed to break.
67. Method of lubrication.
68. Individual adjustment.
69. Two or three bearings for gate stem.
70. Runners.
71. Type.
72. Material.
73. How secured to shaft?
74. Capable of runaway speed.
75. Removable wearing surfaces.
76. At top of runner.
77. On runner.
78. On casing.
79. At bottom of runner.
80. On runner.
81. On casing.
82. Drainage back of runner.
83. Main shaft.
84. Material.
85. Polished.
86. Type of coupling.
87. Coupling bolts.
88. Coupling reamers.
89. Distance center line of turbine to end of shaft.
90. Diameter.
91. Main shaft bearings.
92. Type.
93. Lubrication.
94. Cooling system, piping and screens.
95. Lining.

Governors

96. Type.
97. Capacity, foot-pounds.
98. Servo-motors or levers.
99. Number of servo-motors.
100. Oil pressure and quantity furnished.
101. Speed.
102. Pumping system.
103. Piping.
104. Hand control.
105. Switchboard control.
106. Guaranteed speed regulation.
107. Load-off to zero.
108. ....% load ....% speed change.
109. Load-on from zero.
110. ....% load ....% speed change.
111. Based on what generator WR^2?
112. Dead beat.
113. Will not hunt.
114. Operate in parallel with others.
115. Sensitive to what per cent load change?
116. Speed-limiting device.
117. Air compressor.

118. Water-pressure rise for full load off.
119. Water-pressure drop for full load on.

Accessories

120. Vacuum and pressure gages.
121. Tachometers.
122. Friction brakes.
123. Relief valves.
124. Breaking plates.
125. Penstock valves companion flanges, and bolts.
126. Set of wrenches and wrenchboard.
127. Complete set of cup leathers, packing, etc.
128. All jigs, gages and templates retained for replacements.
129. Handling devices.
130. Jack screws for breaking joints with bronze screw plugs.
131. Foundation bolts.

Workmanship

132. Give here a comparison of workmanship specifications and working stresses used in design.

CHAPTER XXIX

ELECTRICAL DESIGN

BY RAYMOND A. HOPKINS

343. Electrical Design.—The electrical work of a hydro-electric development starts at the coupling which drives the generator, and includes the power-station electrical apparatus and wiring, the transmission line, and in some cases, the receiving and distributing substations. This general sequence will be observed in the following pages on electrical work, except that the treatment of substations will be combined with that of power stations with differences pointed out as they occur in order to avoid repetition.

344. Current.—A current of electricity is conceived of as a stream of moving electrical particles or quantities. The strength of current is measured by the number of unit quantities which pass a given cross-section of a conductor per unit of time. The unit of current is the ampere.

A direct-current circuit is one in which the direction of the current and voltage does not change. An alternating-current circuit is one in which the current and voltage alternate in direction periodically. In America the great bulk of power is generated, transmitted, and utilized as alternating-current power. In commercial power circuits the positive and negative loops are equal to each other and closely approach the sine-wave shape as illustrated by the graph in rectangular coordinates in Fig. 430.

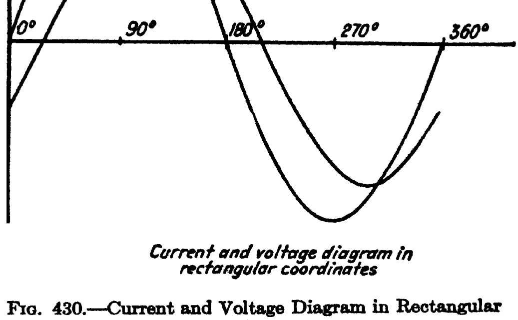


FIG. 430.—Current and Voltage Diagram in Rectangular Coördinates.

345. Voltage.—The flow of current in an electrical circuit is due to a difference of electrical pressure or potential from point to point in the circuit.

The total difference of potential applied to the circuit is called the impressed voltage. The unit of voltage is the volt.

The voltage of a circuit generally refers to the maximum voltage between any two wires, although the voltage to neutral or to ground may be less. A three-phase system has two voltages, i.e., the voltage between wires, known as the " line voltage " or " delta voltage," and the voltage between a wire and neutral, known as the " line to neutral voltage " or " star voltage."

When speaking of a system having a neutral, it is customary to mention both the maximum voltage and the neutral voltage as, for instance, a three-wire, single-phase, 220–110-volt system or a four-wire, three-phase, 4000–2300-volt system.

In order to allow for voltage drop in the distribution system, it is customary to rate generators 4 or 5 per cent higher than motors as, for instance, a 2300-volt generator and a 2200-volt motor.

Nominal voltages in general use in the United States are as follows:

Direct-current

8, 12, 16, 32, 48—Telephones, signals.
115, 230—Lamps, motors, heaters, exciters.
550—Motors, heaters, railways.

Alternating-current

115, 230—Lamps, motors, heaters.
440, 550—Motors, heaters.
2300, 4600, 6600, 11,000, 13,200—Generators, distribution, motors.
22,000, 33,000, 44,000, 66,000, 88,000, 110,000, 132,000, 154,000, 220,000—Transmission.

346. Effective Values.—From Fig. 430 it is seen that the instantaneous values of the current and of the voltage vary continually between maximum positive values and maximum negative values. The effective or root-mean-square (r.m.s.) value is the square root of the mean of the squares of the instantaneous values for one complete cycle. For a sine wave, the r.m.s. value is equal to the maximum value divided by $\sqrt{2}$. The r.m.s. value determines the energy and is always understood except when specifically stated otherwise.

347. Power.—Electrical power is the time rate at which electrical energy or work is expended or delivered. The unit of power is the watt. For a simple resistance circuit, the power is equal to the current multiplied by the voltage, or

$$P = EI \quad \quad (179)$$

348. Energy.—Electrical energy or work is expended in a circuit whenever a voltage is applied to the circuit and a current flows. The unit of energy

is the watt-hour. For a simple resistance circuit, the energy is equal to the product of the current, the voltage, and the time, or

$$W = EIt. \quad \ldots \ldots \ldots \quad (180)$$

349. Frequency.—The frequency of an alternating-current circuit is the number of cycles, or complete reversals, per second, which its current and voltage undergo.

A synchronous generator or motor operates at a fixed synchronous speed depending upon its number of poles and the frequency of the circuit. An induction generator or motor operates at a speed near synchronous speed, the slight difference being called the slip and depending upon the load. The relation between synchronous speed, number of poles, and frequency is given by the equation:

$$s = \frac{120f}{p}, \quad \ldots \ldots \ldots \quad (181)$$

where s = synchronous speed in revolutions per minute;
f = frequency in cycles per second;
p = number of ploes.

Various frequencies from 15 to 133 have been tried and are still to some extent in use. In the United States, 25- and 60-cycle frequencies are standard, with 60 cycles predominating, but a number of important 50-cycle systems exist, such as those in Southern California. In general, the 60-cycle frequency is preferable for lighting since it avoids flicker, and the 25-cycle is somewhat better for rotating machines, especially those of the commutator type. For transmission lines the 25-cycle frequency gives less inductive voltage drop and less charging current, but with synchronous condensers at the receiving end the regulation is very satisfactory with the 60-cycle frequency. For large underground cable systems the 25-cycle frequency has some advantage as it causes less charging-current loss and, in the case of single-conductor lead-sheathed cable, less loss in the sheath. Alternators for 60 cycles are usually less expensive than for 25 cycles, except at low speeds, when their size and cost are greater. Transformers for 60 cycles are considerably less expensive and are more efficient than for 25 cycles.

350. Number of Phases.—The phase of an alternating-current circuit or machine generally refers to the number of phases, as single-phase, three-phase, polyphase. A further explanation of these terms is given in Sec. 357, on alternators, and in Sec. 384, on transformers. A comparison of various distribution systems is given in Fig. 431.

The general tendency is toward the universal use of three-phase circuits for all power systems, although in some localities two-phase systems are so thoroughly established that it is questionable whether they will ever be abandoned.

351. Circuit Constants.—Every electric circuit is characterized by one or more of the constants, resistance, inductive reactance, and condensive react-

ance, which are independent of current and voltage, but depend upon the physical composition and dimensions of the circuit. The resistance determines the energy loss by heat; the inductive reactance determines the energy storage in the surrounding magnetic field; and the condensive reactance

COMPARISON OF DISTRIBUTION SYSTEMS

(BASED ON EQUAL FEEDER LENGTHS AND EQUAL LOADS, AND USING SYSTEM "A" AS A BASE FOR COMPARISON)

SYSTEM	DIAGRAM	DESCRIPTION	CURRENT	COPPER SIZE BASED ON CURRENT DENSITY (SHORT FEEDERS)				COPPER SIZE BASED ON PERCENT VOLTAGE DROP (LONG FEEDERS)			
				COPPER SIZE	TOTAL COPPER WEIGHT	PERCENT VOLTAGE DROP	POWER LOSS	COPPER SIZE	TOTAL COPPER WEIGHT	PERCENT VOLTAGE DROP	POWER LOSS
A		1 Ph or d.c. 2 Wire 110 Volt	100	100	100	1.00	100	100	100	1.00	100
B		1 Ph or d.c. 3 Wire 220–110 Volt	Outside 50 Neutral 0	Outside 50 Neutral 50	75	.50	50	Outside 25 Neutral 25	37.5	1.00	100
C		2 Ph 3 Wire 110 Volt	Outside 50 Common 70.7	Outside 50 Common 70.7	85.3	1.00	85	Outside 50 Common 70.7	85.3	1.00	85
D		2 Ph. 4 Wire 110 Volt	50	50	100	1.00	100	50	100	1.00	100
E		2 Ph. 5 Wire 220–110 Volt	Outside 25 Neutral 0	Outside 25 Neutral 25	62.5	.50	50	Outside 12.5 Neutral 12.5	31.25	1.00	100
F		3 Ph 3 Wire 110 Volt	57.7	57.7	86.5	.865	86.5	50	75	1.00	100
G		3 Ph 4 Wire 191–110 Volt	Outside 33.3 Neutral 0	Outside 33.3 Neutral 33.3	66.7	.50	50	Outside 16.7 Neutral 16.7	33.3	1.00	100

FIG. 431.

determines the energy storage in the surrounding dielectric field. The three constants are defined more specifically by the following equations:

$$R = \frac{\rho l}{a}, \qquad (182)$$

$$X_L = 2\pi f L, \qquad (183)$$

$$X_C = \frac{1}{2\pi f C}, \qquad (184)$$

where R = resistance in ohms;
X_L = inductive reactance in ohms;
X_C = condensive reactance in ohms;
ρ = resistivity in ohms per mil-foot;
l = length in feet;
a = cross-sectional area in circular mils;
π = 3.1416;
f = frequency in cycles per second;
L = inductance in henries;
C = capacity in farads.

When a circuit contains resistance only, the current is in phase with the voltage, as in Fig. 432. If it contains inductive reactance only, the current lags the voltage by 90 degrees as in Fig. 433. If it contains condensive reactance only, the current leads the voltage by 90 degrees, as in Fig. 434. Usually a circuit contains all three constants to some extent, and the amount the current lags or leads the voltage depends upon the relative values of the inductive and condensive reactances. A circuit of this last type is represented in Fig. 430, where the current lags the voltage by θ degrees. For convenience, the difference between the inductive and condensive reactances, which can be considered the net reactance, is called simply the reactance and is designated by the letter X. For uniformity, the inductive reactance is hereinafter considered positive and the condensive reactance negative. The reactance may be either positive or negative, depending on which one of its two components has the larger value, thus:

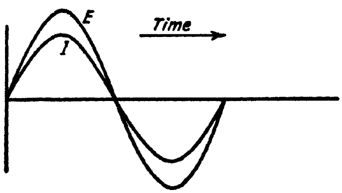


FIG. 432.

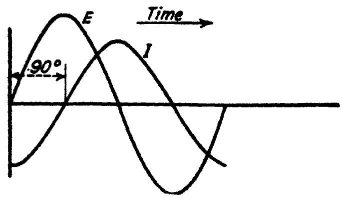


FIG. 433.

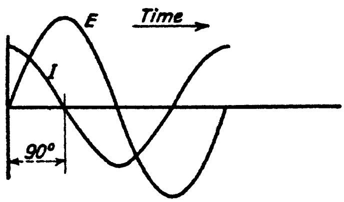


FIG. 434.

$$X = X_L - X_C. \qquad (185)$$

The resistance, R, and the reactance, X, of a circuit, being 90 degrees out of phase with each other as regards their effect on the current, may be resolved into one equivalent value called the impedance, Z, thus:

$$Z = \sqrt{R^2 + X^2}. \qquad (186)$$

The voltage across the terminals of a circuit in terms of the current and the circuit constants is, from Ohm's law:

$$\left.\begin{aligned} E &= IZ \\ &= I\sqrt{R^2 + X^2} \\ &= \sqrt{(IR)^2 + (IX)^2} \\ &= \sqrt{E_1^2 + E_2^2} \end{aligned}\right\}, \qquad (187)$$

where E = voltage in volts;
I = current in amperes;
Z = impedance in ohms;
R = resistance in ohms;
X = reactance in ohms;
$E_1 = IR$ = power component of E;
$E_2 = IX$ = reactive component of E.

Likewise, the current may be expressed in terms of the voltage and reciprocals of the circuit constants as follows:

$$\left.\begin{aligned} I &= EY \\ &= E\sqrt{G^2 + B^2} \\ &= \sqrt{(EG)^2 + (EB)^2} \\ &= \sqrt{I_1^2 + I_2^2} \end{aligned}\right\}, \qquad (188)$$

where, I = current in amperes;
E = voltage in volts;
$Y = \frac{1}{Z}$ = admittance in mhos;
$G = \frac{R}{Z^2}$ = conductance in mhos;
$B = \frac{X}{Z^2}$ = susceptance in mhos;
$I_1 = EG$ = power component of I;
$I_2 = EB$ = reactive component of I.

Resistance, reactance, and impedance are usually more convenient for use with series circuits, while conductance, susceptance, and admittance are more convenient where parallel circuits are involved.

352. Power Factor and Reactive Factor.—The cosine of the angle by which the current leads or lags the voltage is called the power factor of the circuit. The power is the product of the current, voltage, and power factor, thus:

$$P = EI \cos \theta. \qquad (189)$$

The sine of the angle by which the current leads or lags the voltage is called the reactive factor of the circuit. The reactive kv-a. is the product of the current, voltage and reactive factor, thus:

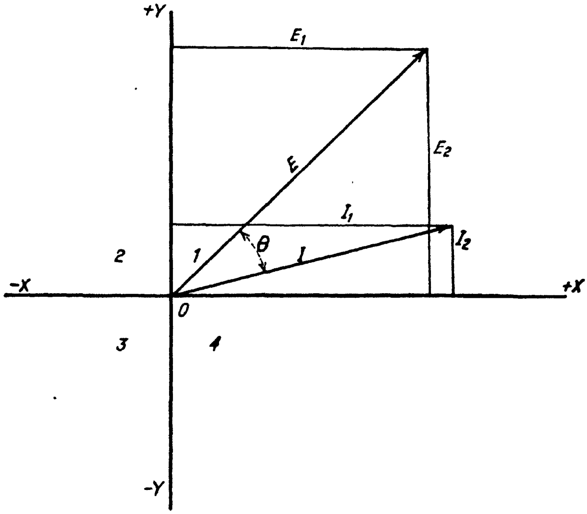
$$Q = EI \sin \theta. \qquad (190)$$

The kilovolt-amperes, or kv-a. of a circuit is the product of the current and voltage, thus:

$$K = EI. \qquad (191)$$

353. Vector Representation.—Alternating currents and voltages are conveniently represented by vector diagrams, as illustrated in Fig. 435, which

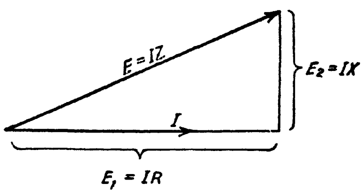
represents the same currents and voltages as are represented in rectangular coordinates in Fig. 430. The length of the vector corresponds to the effective



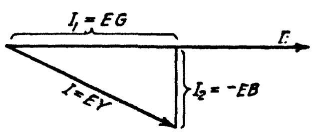
Current and voltage vector diagram

FIG. 435.

value of the current, or voltage, and the angular position represents its phase displacement with reference to axis $0 + X$. The rotation is considered as counter-clockwise, so that in Fig. 435, as in Fig. 430, the voltage leads the current by the angle θ. Each vector is composed of two components, one along the X axis, the other along the Y axis. Components measured to the right of YY or above XX are positive, while those measured to the left of YY or below XX are negative. For convenience, the four quadrants are always numbered in the order shown in Fig. 435. Two or more vector quantities may be added or subtracted by adding or subtracting their components. The relations given analytically in Sec. 351 are shown graphically in Fig. 436.



Voltage vector diagram



Current vector diagram

FIG. 436.

354. Complex Algebra. — Calculation of the performance of electrical circuits involving vector quantities is greatly facilitated by the use of complex

algebra, the principles of which are covered in most textbooks on algebra, and very completely in Dr. C. P. Steinmetz' "Engineering Mathematics." Eq. 186 is expressed in complex notation in the following form:

$$\underset{\cdot}{Z} = R + jX. \qquad (192)$$

The dot under the Z indicates that Z is a vector quantity composed of two components at right angles to each other, namely R and X. The R part of the right-hand member is measured along the X axis and is called the real part of the quantity. The X part, since it is preceded by the symbol j, is measured along the Y axis and is called the imaginary part of the quantity. The symbol j, in addition to indicating measurement along the Y axis, also has the value $\sqrt{-1}$, or $j^2 = -1$. Addition, subtraction, multiplication, and division of complex quantities are performed in the same manner as with ordinary algebraic quantities, by observing this twofold significance of the symbol j.

355. Short-circuit Analysis.—An analysis of the short-circuit characteristics of a power system is essential for determining (1) the interrupting duty for which the circuit-breakers must be designed, (2) the mechanical stresses which the insulators must withstand, (3) the thermal capacities required for current transformers and other series equipment, and (4) proper relay settings. The value of short-circuit current due to a fault, that is, a short circuit or ground, at any point on a system depends upon (1) the connected capacity in synchronous generators and motors, (2) impedances of the synchronous machines, and (3) impedances of transformers, reactors, and circuits between the machines and the fault.

In most cases the resistances are small compared with the reactances, and sufficient accuracy can be obtained by neglecting the resistances and assuming that all reactances are in phase with each other. The error is on the safe side, as the short-circuit current found by this method is somewhat greater than when actual impedances are used.

Calculating tables are used extensively for making short-circuit analyses on the basis of reactances only, and by their use very complicated networks can be analyzed with comparative ease. The calculating table consists of a number of resistances that can be interconnected so as to form a miniature network equivalent to the one under study, together with means of passing current through the miniature system and measuring the values at various points. In the general board, as used by consulting engineers, the resistances are variable and an extensive arrangement of buses, jacks, and cords is provided so that various power systems may be set up with minimum effort. In the special boards used by operating companies, most of the resistances are of fixed values to represent permanent equipment and lines on the system. In all cases the resistances are calibrated in terms of reactance.

The analytical solution of a network, where resistance is neglected and reactances are assumed in phase with each other, consists of replacing element after element by equivalent elements of simpler construction so as finally to resolve the system into one reactance connecting the synchronous equipment

with the fault. A typical system is shown in Fig. 437. The reactance values used apply to one conductor of the three-phase circuit.

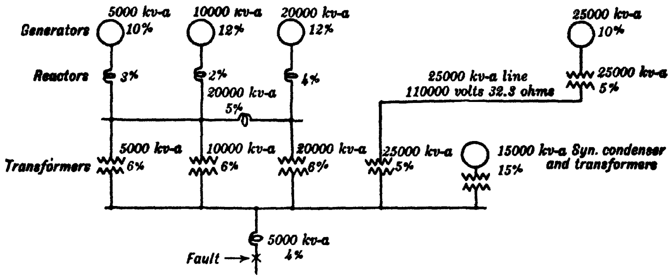


FIG. 437.—System Reactance Diagram.

The first step is to replace the generators, motors, reactors, transformers, and circuits by their equivalent reactances, giving Fig. 438. All reactances must be expressed in the same terms, that is, either in ohms at a common voltage base, or in percentages to a common kv-a. base. When the latter method is used the common base is usually arbitrarily chosen as 100,000 kv-a., or as the total connected kv-a. synchronous capacity. The following relations are useful in reducing the various elements of the system to a common base:

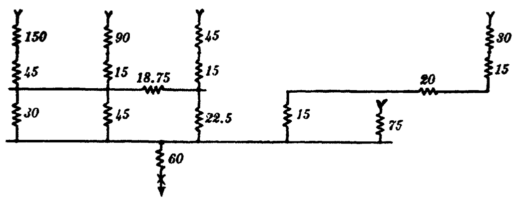


FIG. 438.—Equivalent Reactance Diagram. (All reactances in percentages to 75,000 kv.-a. base.

$$\frac{\text{Reactance in percentage}}{\text{Reactance in ohms} \times 100} = \frac{\text{Rated volt-amperes}}{(\text{Rated volts})^2} \quad (193)$$

$$\frac{\text{Reactance in percentage to base } K_1}{\text{Reactance in percentage to base } K_2} = \frac{K_1}{K_2} \quad (194)$$

$$\frac{\text{Reactance in ohms at voltage } E_1}{\text{Reactance in ohms at voltage } E_2} = \frac{E_1^2}{E_2^2} \quad (195)$$

Subsequent steps in reducing the network are shown in Fig. 439. The working tools are explained by the elementary diagrams of Fig. 440. Two very useful rules for reducing networks and avoiding the use of reciprocals are as follows:

1. Series reactances may be combined by reducing to the same base and adding percentages.
2. Parallel reactances may be combined by reducing to the same percentage and adding bases.

For cases where these simple substitution processes are not adequate to reduce the network to a single reactance, it becomes necessary to set up a series of differential equations based on Kirchhoff's laws and solve them by substitution.[1]

By dividing the rated current per conductor by the per cent reactance, found as described above and expressed as a decimal, the initial value of short-

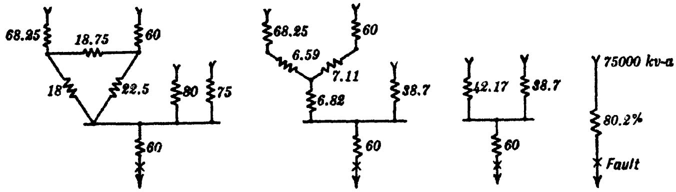


FIG. 439.—Simplification of Typical Network.

circuit current is obtained. In case the reactance has been worked out in ohms, the initial value of short-circuit current can be obtained by dividing the rated voltage to neutral by the ohms. The value obtained by either method is known as the initial r.m.s. symmetrical short-circuit current per conductor. If the short circuit occurs at the instant the voltage wave is passing through zero, the short-circuit current wave will be offset about its

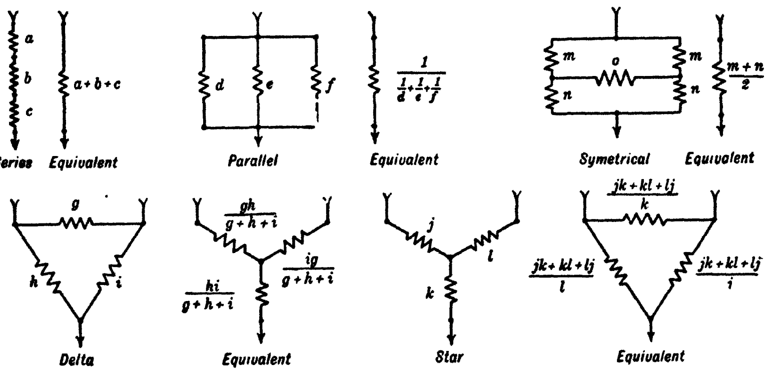


FIG. 440.—Elements of Simplification of Networks.

axis, the initial r.m.s. unsymmetrical current being approximately 1.73 times the symmetrical current. The armature reaction of the synchronous machines rapidly reduces the initial value of short-circuit current, the final, steady value being known as the sustained short-circuit current.

For the application of oil circuit-breakers, advantage is taken of the fact

[1] Analytical Solution of Networks, by R. D. Evans. The Electric Journal, Vol. XXI, pp. 149, 207, April and May, 1924.

that the short-circuit current decreases during the first few seconds after the instant of short circuit, and hence the value of current which the breaker will be called upon to interrupt will depend not only upon the initial value of short-circuit current, but also upon the time elapsed between the short circuit and the parting of the breaker contacts. As the result of innumerable tests and oscillographic records, the manufacturers of synchronous machines have obtained current-decrement factors of standard machines, which are

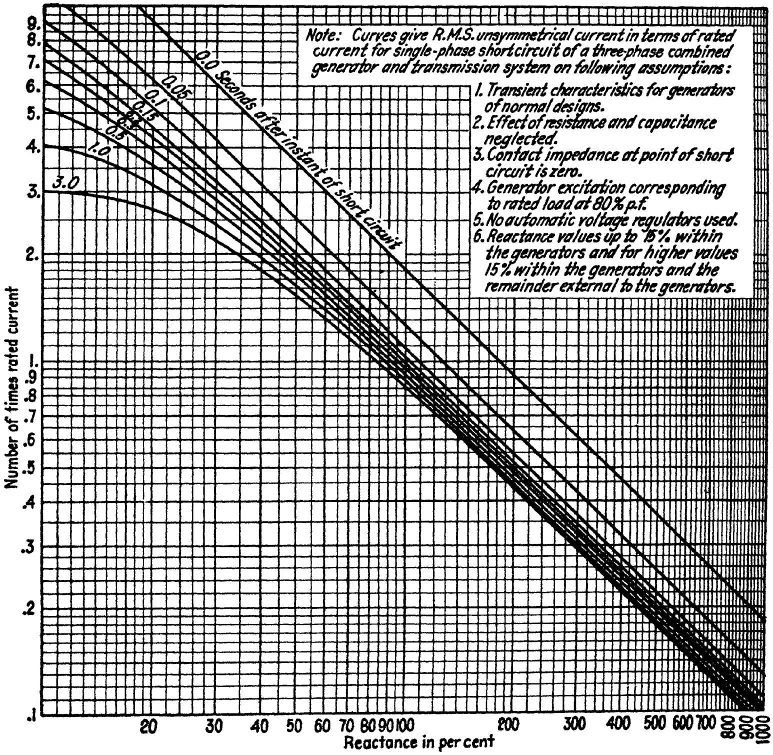


FIG. 441.—Short-circuit Current Decrement Curves

given in Fig. 441. The value of r.m.s. unsymmetrical short-circuit current in amperes at moment of opening, which is the basis of circuit-breaker rating, is obtained by multiplying the rated current of the system, corresponding to the kv-a. base at which the reactance of the system is expressed, by the factor taken from the curve. In applying the decrement factors, any branches of the system that feed directly into the fault should be treated separately. For instance, in Fig. 437, for a fault occurring on the bus instead of on the feeder beyond the reactor, the entire system should be treated as three separate branches: (1) the 5000, 10,000, and 20,000 kv-a. generators; (2) the

25,000 kv-a. generator, and (3) the 15,000 kv-a. condenser. Each of these branches should be reduced to a single reactance based on the kv-a. of the branch, and after the decrement curves have been applied to each, the resulting three short-circuit currents should be added to obtain the total short-circuit current into the fault.

The analysis outlined above is intended to cover three-phase short circuits or single-phase line-to-line short circuits. As the latter have less decrement than the former they represent a more severe circuit-breaker duty and are used for the selection of breakers. For this reason the curves of Fig. 441 are drawn to apply to single-phase line-to-line short circuits. Under some conditions of neutral grounding, the single-phase line-to-neutral short circuit is even more severe than the single-phase line-to-line and should be investigated.[2]

For the determination of the mechanical forces to which bus supports and insulators will be subjected when a short circuit occurs, consideration must be given to the initial peak unsymmetrical value of current in at least one phase. The generally accepted value is equal to $1.8\sqrt{2}$ times the initial r.m.s. symmetrical value. In equations 197, 198 and 199 the r.m.s. symmetrical values are used, but the effect of the peak unsymmetrical current in one phase is included in the constants. The notation is as follows:

F = force, in pounds per foot of conductor. The force between two conductors is attraction when the currents are in the same direction, and repulsion when in opposite directions;
S = spacing between centers of conductors in inches;
$i_1 i_2$ = simultaneous values of currents in the two conductors;
I_0 = initial r.m.s. symmetrical short-circuit current in amperes.

The fundamental equation giving the force acting between two parallel conductors is:

$$F = 5.4 \frac{i_1 i_2}{S \times 10^7} \quad \quad (196)$$

For a single-phase, line-to-line short circuit on a single-phase system, or on a three-phase system with conductors arranged in any configuration, the force is repulsion acting in the plane of the two conductors under short circuit. The maximum value is:

$$F = 35 \frac{I_0^2}{S \times 10^7} \quad \quad (197)$$

For a three-phase, line-to-line short circuit on a three-phase system with conductors arranged at the vertices of an equilateral triangle, the maximum force acts away from the center of the triangle and is:

$$F = 30.3 \frac{I_0^2}{S \times 10^7} \quad \quad (198)$$

[2] Single-phase Short-circuit Calculations, by W. W. Lewis. General Electric Review, July, 1925, p. 479. Calculations of Single-phase Short Circuits, by the Methods of Symmetrical Components, by A. P. Mackerras. General Electric Review, April, 1926, p. 218.

For a three-phase, line-to-line short circuit on a three-phase system with conductors arranged in a plane, the forces act in the plane of the conductors and the maximum value of the force on any conductor is:

$$F = 26.3 \frac{I_0^2}{S \times 10^7} \quad \quad (199)$$

Equations 197, 198 and 199 assume round or square conductors and do not take account of the natural frequency of vibration of the conductors and supports. H. B. Dwight [3] has shown that with conductors of rectangular section the force varies slightly from the above, depending upon the shape and spacing of the conductors. The variation is greatest for very close conductor spacings, but for ordinary spacings used in power station wiring the variation is negligible. O. R. Schurig and M. F. Sayre [4] have developed a method of taking account of the natural frequency of the conductors and supports and have shown that under certain conditions the stresses on supports may exceed those given by the above equations. For general purposes, the above equations may be safely used by applying a factor of safety of 2 to the values obtained and specifying that the supports shall safely withstand this stress applied in any direction at the center line of the conductor.

356. Energy-dissipating Rheostats.—Rheostats of large capacity are often desired for loading generators while under test, and are sometimes installed permanently to regulate speed, to ground the neutral, or for other purposes.

Submerged-wire rheostats consist of coils of wire submerged in water which acts as a cooling medium. They are very satisfactory for voltages up to 500 volts. The coils may be arranged in a barrel or tank or may be submerged in the river. Adjustment of load may be obtained by the use of taps or interconnections.

Electrolytic rheostats consist of metal plates or electrodes immersed in a solution or electrolyte which acts as a conducting medium. Such rheostats are often used with voltages up to 1000 volts. Various electrodes, such as lead, carbon, copper, or iron, and electrolytes, such as acids, bases, and salts, may be used, provided the combination does not produce chemical action. Adjustment of load may be obtained by varying the distance between electrodes, the depth of immersion, or the strength of the electrolyte.

Water rheostats consist of terminals, usually plates or pipes, immersed in water which actsas a conducting medium. Water rheostats are adaptable to all voltages above 1000 volts. The terminals may be arranged in a tank but are more commonly placed directly in the river. Adjustment of load may be obtained by varying the spacing of terminals or the depth of immersion.

[3] Repulsion between Strap Conductors, by H. B. Dwight. Electrical World, Vol. 70, p. 522, April, 1925.

[4] Mechanical Stresses in Bus-bar Supports During Short Circuits, by O. R. Schurig and M. F. Sayre. Journal A. I. E. E., Vol. 44, p. 365, April, 1925.

CHAPTER XXX

GENERATORS, EXCITERS AND TRANSFORMERS

BY RAYMOND A. HOPKINS

357. Generators.—A generator is defined as a machine that transforms mechanical energy into electrical energy. The essential features are (1) a field or assembly of magnets arranged to produce a magnetic flux, and (2) an armature or assembly of electric conductors arranged across the path of the magnetic flux. The field and armature are so mounted that by the application of mechanical force a relative motion is produced between the magnetic flux and the electric conductors, and this motion induces in the conductors an electromotive force. Since the field poles are arranged alternately, positive and negative, around the periphery of the generator, the polarity of the electromotive force induced in the armature is alternating.

The alternating-current synchronous generator, or alternator, delivers its induced alternating current directly to the external circuit. Alternating-current generators are used in hydro-electric stations where the output is to be transmitted over long lines, since the alternating current can be transformed up to suitable transmission voltage. They are also generally used where the output is to be distributed locally, since the greater part of power and lighting service is alternating.

The direct-current generator is provided with a commutator which rectifies its induced alternating current and delivers direct current to the external circuit. Direct-current generators are occasionally used where the output is to feed local railway or industrial loads which require direct current. Small direct-current generators, called exciters, are required in all power stations for energizing the magnetic fields of the alternators.

The induction generator is similar, physically, to an induction motor with a squirrel-cage secondary winding. Instead of revolving at slightly less than synchronous speed, as an induction motor does when consuming electrical energy, the induction generator is driven slightly above synchronous speed and, as a result, delivers electrical energy. In common with the induction motor, however, the induction generator consumes a lagging exciting current, which in most cases is detrimental to the system, but in some cases can be used to advantage to offset in part the leading exciting current consumed by the transmission lines. Advantages of the induction generator as compared with the synchronous generator include less short-circuit current, more rugged rotor construction, less danger of overvoltage resulting from over-speed, and

less danger of overload. Disadvantages include smaller air gap, higher cost in lower speeds, and lagging exciting current, as mentioned above.

The single-phase generator has one armature winding arranged to deliver a single-phase alternating current to a two-wire system. The center point of the winding is sometimes brought out to a third terminal and can be used as a grounding point or as a neutral for a three-wire system if desired. Single-phase generators are only occasionally used to supply special loads, such as single-phase railways, electrochemical and electrothermal industries, and in smaller sizes for lighting loads. The arrangement of the single-phase cores and windings is inherently uneconomical, with the result that a single-phase generator is about 30 per cent larger than a two- or three-phase generator of equal capacity.

The two-phase generator has two armature windings arranged to deliver two single-phase alternating currents, 90 electrical degrees apart in phase, to a four-wire system; hence the machine is sometimes called a quarter-phase generator. The center points of the two windings are sometimes connected together, and then the generator is said to be interconnected. The point of interconnection may be used as a grounding point or as a neutral for a five-wire system if desired. In case the machine is not interconnected, two of the terminals, one from each winding, may be connected together so that the machine can be used on a three-wire system. The common connection can be grounded if desired. Two-phase generators are frequently used in territories where two-phase systems are well established, but, owing to the better transmission economy of the three-phase system, it is often preferable to use three-phase generators even if phase converters are necessary.

The three-phase generator has three armature windings arranged to deliver three single-phase alternating currents, 120 electrical degrees apart in phase, to a three-wire system. The delta-connected, three-phase generator has the end of each winding connected to the beginning of the next winding, the three points of connection being used as line terminals. The star or Y-connected machine has one end of each winding connected together in common, the other three ends being used as line terminals. The common connection may be used as a grounding point or as a neutral for a four-wire system if desired. The delta connection is sometimes used for low-voltage, high-amperage machines, but usually the star winding is preferred, because of the higher voltage between terminals as compared with the voltage of each winding, and because of the better opportunity to apply relay protection. Three-phase generators are always preferred to two-phase or single-phase machines except where the latter are needed to meet the local conditions mentioned above.

The interconnections between windings, described above, are sometimes made within the machine by the manufacturer, but with the larger machines it is usual for the manufacturer to bring out both ends of each winding to terminals. The interconnections are then made in the station and are arranged to include the necessary current transformers for relay protection.

358. Construction.—A cross-sectional view through the axis of a typical vertical, revolving-field type, moderate-speed, water-wheel driven alternator

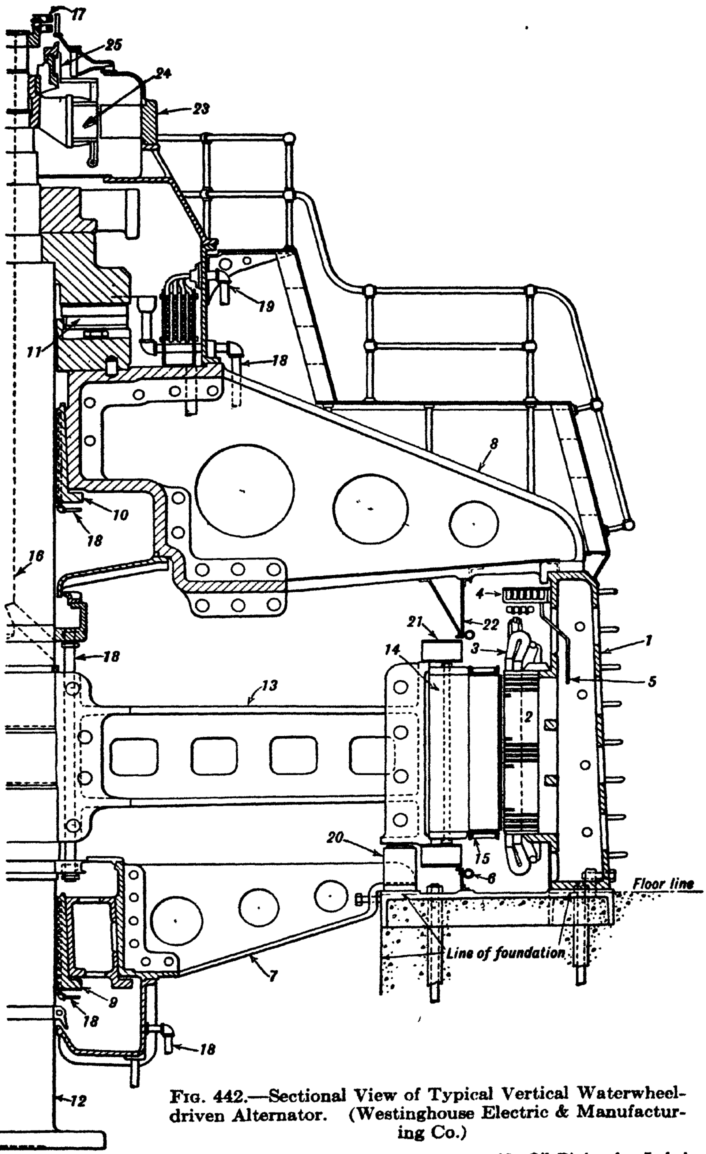


Fig. 442.—Sectional View of Typical Vertical Waterwheel-driven Alternator. (Westinghouse Electric & Manufacturing Co.)

1. Stationary Armature Frame
2. Laminated Armature Core
3. Armature Winding
4. Armature End Connections
5. Armature Leads
6. Fire Extinguisher Water Piping
7. Lower Bearing Bracket
8. Upper Bearing Bracket
9. Lower Bearing
10. Upper Bearing
11. Thrust Bearing
12. Shaft.
13. Revolving Field Frame
14. Laminated Field Core
15. Field Winding
16. Duct through Shaft for Field Leads
17. Collector Rings
18. Oil Piping for Lubricating Bearings
19. Water Piping for Cooling Bearings
20. Brake and Jack
21. Fans
22. Air Baffles
23. Exciter Field
24. Exciter Armature.
25. Exciter Commutator

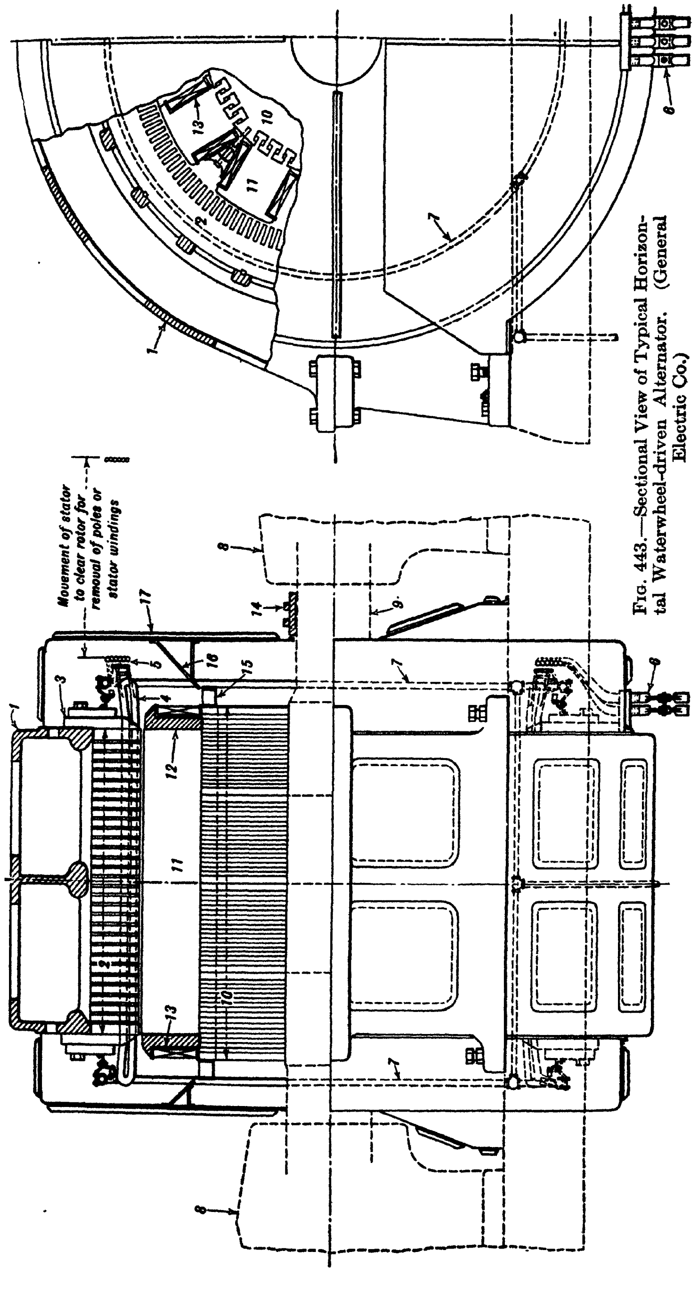


FIG. 443.—Sectional View of Typical Horizontal Waterwheel-driven Alternator. (General Electric Co.)

1. Stationary Armature Frame
2. Laminated Armature Core
3. Stator Clamping Flange
4. Armature Winding
5. Armature End Connections
6. Armature Terminals
7. Fire Extinguisher Water Piping
8. Bearings
9. Shaft
10. Field Spider
11. Laminated Field Core
12. Rotor End Plates
13. Field Winding
14. Collector Rings
15. Fans
16. Air Deflectors
17. End Shield

is shown in Fig. 442. Fig. 443 shows a somewhat higher-speed horizontal machine. The revolving-field type of alternator has become practically standard for the following reasons: (1) the armature windings, which carry heavy currents and must be insulated for relatively high voltages, are freer from mechanical strains and vibrations when built in the form of a stator, while the low-voltage field windings are more adaptable to rotor construction; (2) the problem of taking off the large, high-voltage currents from a rotating armature would be very serious, but the lighter field currents can be handled very satisfactorily by slip rings and brushes.

Horizontal-shaft and vertical-shaft alternators are essentially the same electrically, being merely modified mechanically to suit the particular type of construction. The vertical type is better adapted to larger capacities at low speeds on account of the large periphery required to obtain a sufficient number of poles. The position of the shaft is generally chosen, however. so as to obtain the most favorable water-wheel design.

The rotating field, or rotor, is of the distributed-pole type when the windings are embedded in slots in the face of the cylindrical wheel so that the pole pieces are integral with the wheel, and is of the salient-pole type when the windings surround individual pole pieces mounted on the periphery of the wheel. The former type is used for the highest-speed generators on account of its greater strength and compactness, but the majority of water-wheel generators have salient-pole type fields. High-speed wheels are solid, and are built up of steel plates or forgings. Slow-speed wheels are of large diameter and take the form of cast-iron spiders with spokes. Since water wheels are subject to runaway speeds of 150 to 200 per cent of normal speed, it is necessary that water-wheel-driven generator rotors be designed to withstand safely the mechanical stresses due to approximately double normal speed.

Field poles are laminated or built up of thin sheets of magnetic steel so as to prevent power losses by hysteresis and eddy currents. The coils are form-wound and are slipped over the poles before assembly. Round, square, or rectangular copper is used, as is best adapted to the space available and the current-carrying capacity required.

Slip rings and brushes are used to conduct the field current to the coils on the rotor. The rings are usually of cast iron and the brushes of carbon. The brushes are held in contact with the rings by means of springs and are usually staggered to prevent their wearing grooves in the rings.

The stationary armature, or stator, consists of a circular steel frame supporting the core and coils. The core, like the field poles, is laminated to prevent losses. The coils are form-wound and are laid in slots in the core. The ends of the coils, which project beyond the core, must be substantially braced to prevent distortion by the mechanical stresses accompanying heavy loads and short circuits.

Bases are generally supplied for supporting and accurately alining the stator frame and the bearings. In the larger sizes of horizontal machines, four pads or sole plates are often supplied for grouting in the foundations to support the two sides of the stator frame and the two bearings, instead of a

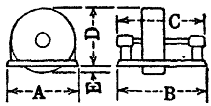
single base. Smaller machines generally have the bearings built as a part of the end shields and do not have bases and bearing pedestals.

359. Weights and Dimensions.—Approximate weights and dimensions of typical standard water-wheel-driven generators without direct-connected exciters are given in Tables LVII and LVIII. Since the power developed is proportional to the product of the magnetic field strength and the speed with which the field cuts the armature conductors, it is evident that the slower the speed, the larger the size and weight must be for a given power rating. The weight of the heaviest piece, which often determines the capacity of the turbine-room crane, is approximately 40 per cent of the total net weight for horizontal machines and 35 per cent for vertical machines.

TABLE LVII

Approximate Weights and Dimensions Horizontal Water-Wheel Generators

This table applies to three-phase, 60-cycle, 80% p. f. alternators without direct connected exciters.



Rating, Kv-a.	Speed, R.P.M.	Flywheel, Effect, WR^2	Weight in Pounds		Dimensions in Inches				
			Shipping	Net	A	B	C	D	E
1,000	600	9,600	15,700	13,070	79	78	88	69	5
1,000	300	42,300	25,000	20,840	117	83	104	94	14
1,000	120	216,000	42,000	34,980	178	89	105	131	29
2,000	600	26,000	32,500	27,200	99	95	108	82	8
2,000	300	72,000	41,000	34,060	117	110	128	94	14
2,000	150	300,000	56,500	47,020	178	104	126	131	29
5,000	600	90,000	71,000	59,440	118	156	165	99	9
5,000	300	360,000	87,000	72,880	163	120	144	125	23
5,000	150	900,000	115,000	95,340	209	128	158	140	50
7,000	360	330,000	100,000	83,000	144	113	142	109	25
10,000	150	2,800,000	225,000	190,460	259	124	169	169	61
12,000	400	750,000	200,000	165,400	167	127	162	124	28
17,500	360	1,120,000	265,000	222,600	203	177	202	126	42
20,000	360	1,400,000	300,000	250,200	208	189	225	126	42
22,500	200	5,500,000	418,000	348,500	268	178	240	186	66

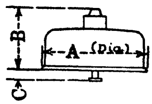
Data from General Electric Company.

360. Rating.—Although the complete rating of an alternator as given on its rating plate includes output, power factor, voltage, phase, frequency, speed, and current, the term rating usually applies specifically to guaranteed output. The maximum output is limited by the allowable maximum temperature which the insulation will safely withstand. As it is difficult to measure the tempera-

TABLE LVIII

APPROXIMATE WEIGHTS AND DIMENSIONS VERTICAL WATER-WHEEL GENERATORS

This table applies to three-phase, 60-cycle, 80% p. f. alternators without direct-connected exciters.



Rating, Kv-a.	Speed, R.P.M.	Flywheel Effect, WR^2	Weight in Pounds		Dimensions in Inches		
			Shipping	Net	A	B	C
1,250	120	410,000	89,700	78,000	159	145	38
1,875	225	250,000	79,300	69,000	159	103	26
1,875	100	500,000	103,000	89,400	177	115	42
2,250	120	665,000	115,000	100,000	177	115	33
3,000	200	300,000	103,100	89,500	159	111	33
5,000	277	600,000	129,000	112,000	177	115	45
6,500	200	1,500,000	210,000	182,500	196	147	34
7,500	327	1,100,000	197,000	171,100	177	145	34
12,000	240	2,000,000	237,000	206,000	196	149	54
12,000	150	4,500,000	345,000	300,000	240	178	49
14,000	180	4,500,000	418,000	362,500	243	178	44
17,000	360	1,600,000	330,000	287,000	177	200	44
25,000	100	24,000,000	816,000	710,000	348	180	44
30,000	514	2,000,000	502,000	437,000	177	218	42
30,000	257	6,250,000	589,000	510,600	217	185	43

Data from Westinghouse Electric and Manufacturing Company.

ture of the hottest spot in the machine, a conventional " hottest-spot " allowance of 5° to 15° C. is subtracted from the maximum allowable temperature to obtain the guaranteed observable temperature. As the hottest-spot allowance depends upon the method of measuring the temperature, it is necessary to specify the method. The observable temperature is stated in Centigrade degrees rise above an ambient or room temperature of 40° C. and is usually specified for continuous operation, meaning operation continued long enough for the machine to attain its maximum temperature. Standardized temperatures for various kinds of insulation are given in the Standards of the A.I.E.E. As the heating depends largely upon the current, which varies inversely as the power factor, the rating of an alternator is always given in kilovolt-amperes.

Alternators are commonly rated at 80 per cent power factor, although if desired they may be rated at any other power factor that best suits conditions of prospective load. If operated at a lower power factor than that for which it is rated, the alternator will not carry rated kv-a. load without exceeding rated temperature rise; but if operated at a higher power factor than that for

which it is rated, it will carry more than rated kv-a. load without exceeding rated temperature rise. This is because at a low power factor the field current, and consequently the total heating, is greater than for the same kv-a. load at a higher power factor. The fact should not be overlooked, however, that the prime mover is generally designed for a maximum output corresponding to rated generator kilowatt output and cannot efficiently develop more, even though the alternator may. For example, a 25,000 kv-a., 80 per cent power factor generator can deliver only 20,000 kw. at 80 per cent power factor, but can easily deliver 25,000 kw. at 100 per cent power factor. Its prime mover, however, can probably only develop a maximum of 20,000 kw. at good efficiency.

The terminal voltage of an alternator is determined by the arrangement and connection of the armature conductors, the speed of rotation, and the field strength. The voltage can be adjusted within limits by adjusting the field strength, and field rheostats are provided for this purpose. It is usually advisable for the purchaser to specify that the generator shall be capable of successful continuous operation at from 95 per cent to 105 per cent of rated voltage at rated kilovolt-amperes, power factor, and speed, and with not more than rated excitation voltage applied at the field terminals. Alternators are ordinarily wound for any standard voltage up to about 15,000 volts, this limit being practically set at present by cost of insulation and risk of failure. It is sometimes considered advisable to use a lower voltage, such as 6600 or 11,000 volts, so as to enjoy the added security of a more conservative design.

An alternator operates at a fixed synchronous speed depending upon its number of poles and the frequency of the circuit to which it is connected. It is, therefore, necessary that the number of poles in the alternator and the speed of the water wheel be carefully selected with reference to the frequency of the prospective load. (See Sec. 349.)

361. Efficiency.—The true efficiency of a generator is the ratio of its useful output to its total input. The determination of the true efficiency involves the accurate measurement of the output and simultaneous input or the accurate measurement of all the losses. Accurate determination by either of these methods is impossible without the use of highly refined laboratory instruments and means of driving the machine at full load and absorbing its output. Consequently a "conventional efficiency" which is readily obtained by measuring the principal losses with commercial instruments, and which is very close to the true efficiency, is universally used. The conventional efficiency of an alternator is defined in the Standards of the A.I.E.E. and takes into account (1) core loss, (2) I^2R loss in armature, field, and field rheostat, (3) stray-load loss, (4) windage and bearing-friction losses, and (5) brush-friction and brush-contact losses. Sometimes rheostat losses are not included, on the assumption that the exciter voltage will be varied to suit the field requirements. The efficiency depends upon the size, speed, and general design. Higher than average efficiencies can usually be obtained, at greater first cost, by careful design, and the expense is sometimes warranted by service conditions. Approximate efficiencies of typical standard alternating-current generators are given in Table LIX.

TABLE LIX

APPROXIMATE EFFICIENCIES OF STANDARD WATER-WHEEL GENERATORS, THREE-PHASE, 60-CYCLE, 80 PER CENT P. F.

These efficiencies take into account the following losses: Core, armature I^2R, field, I^2R, field rheostat I^2R, stray-load, windage, bearing-friction, brush-friction, brush-contact. Exciter losses are not included.

Rating, Kv-a.	Speed, R.P.M.	Efficiency at 100% P.F.			Efficiency at 80% P.F.		
		50% Load	75% Load	100% Load	50% Load	75% Load	100% Load
100	360	90.1	91.6	91.2	86.4	87.7	88.0
	1200	90.7	92.2	93.1	87.8	89.2	89.2
1,000	150	91.2	92.8	93.7	89.9	90.8	91.3
	600	92.5	94.2	95.0	91.0	92.7	93.7
5,000	150	94.5	95.7	96.5	93.7	95.0	95.5
	600	94.8	96.2	97.0	94.3	95.5	96.2
10,000	150	94.7	96.1	96.8	94.1	95.4	96.0
	400	94.8	96.2	96.9	94.3	95.5	96.2
20,000	120	94.7	96.1	96.8	94.1	95.4	96.0
	360	95.2	96.5	97.4	94.7	96.1	96.8
30,000	120	94.9	96.3	97.1	94.3	95.7	96.4
	360	95.1	96.4	97.3	94.6	95.9	96.7

362. Regulation.—The regulation of an alternator is defined in the Standards of the A.I.E.E. as the rise in voltage, expressed in per cent of normal rated load voltage, that occurs when the specified load at specified power factor is reduced to zero. It is usual to specify the load as rated load and to give the regulation at 100 per cent and 80 per cent power factors. A direct method of obtaining the regulation of an alternator is by loading it to the specified load and power factor and then reducing the load to zero, measuring the voltage before and after removing the load, with speed and excitation adjusted to the same values for both measurements. This method is not always applicable to large alternators on account of difficulty in obtaining the desired loads and power factors, and graphical methods are, therefore, used as described in Sec. 363 and in the Standards of the A.I.E.E.

363. Characteristic Curves.—The typical characteristic curves of an alternator are shown in Fig. 444. The curves usually obtained by test or calculated in preliminary design are (1) no-load saturation, (2) air gap, (3) short-circuit synchronous impedance, and (4) zero power factor. The no-load saturation curve shows the relation between field excitation and armature voltage at rated speed with armature open-circuited. The straight lower part indicates that with low values of excitation the voltage is proportional to the excitation. The curved upper portion indicates that as the excitation is increased above a certain point the iron becomes saturated and the voltage is no longer proportional to the excitation. The air-gap curve is merely the

extension of the straight part of the saturation curve and does not take account of the saturation of the iron. The short-circuit synchronous-impedance curve shows the relation between field excitation and armature current at rated speed with armature short-circuited at its terminals. The curve is usually a straight line because the demagnetizing effect of the heavy armature current opposes the magnetizing effect of the field excitation and as a result there is little or no saturation. This demagnetizing effect always exists when the armature carries current and is known as armature reaction. The zero

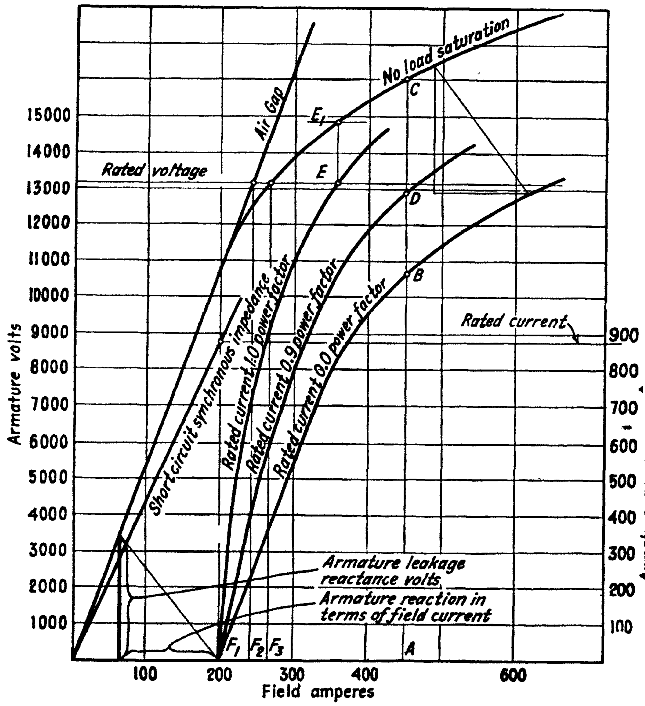


FIG. 444.—Alternator Characteristic Curves. 20,000 kv.-a. 13,200 v. 875 amp. 90 per cent P. F. 3 Ph. 60 cy.

power-factor curve shows the relation between field excitation and armature voltage at rated speed with armature carrying rated current at zero power factor. The characteristic is sometimes obtained by loading the machine with an underexcited, unloaded, synchronous motor. The characteristic curves for various power factors may be obtained graphically as shown in Fig. 445. The method ignores resistance, which is usually negligible with respect to reactance. To obtain the 90 per cent power-factor curve, draw lines *oa* and *ob* perpendicular to each other, and *oc* making with *ob* an angle, θ, whose cosine is 0.9. On the characteristic-curve sheet, select any field

current, as A, and read voltages AC and BC. Lay off on line oc, Fig. 445, voltage $od = BC$, and with d as a center and AC as a radius cut oa at e. Returning to the curve sheet, locate point D by making $AD = oe$. Repeat for other values of field current until a sufficient number of points on the curve are obtained. Curves for other power factors may be obtained by a like process, making θ equal to $\cos^{-1}$ power factor. For 100 per cent power factor line oc will coincide with ob. It will be seen that for rated current and voltage at 90 per cent power factor a field current of 465 amperes is required.

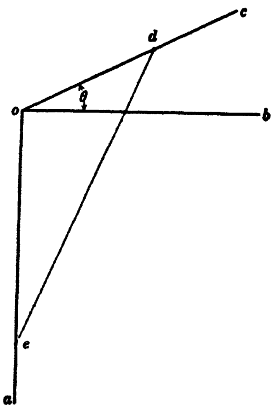


FIG. 445.—Graphic Method of Obtaining Characteristic Curves at Various Power Factors.

The armature reaction in terms of field current and the armature leakage reactance volts may be obtained graphically, referring to Fig. 444, by finding by trial the right-angle triangle of such shape that when moved parallel to its lower position its two acute corners will trace the no-load saturation and zero power-factor curves respectively. For the generator shown, the armature reaction in terms of field current is 135 amperes and the leakage reactance is 3400 volts, or 25.8 per cent.

The synchronous reactance is defined as the voltage drop, in per cent of rated voltage, caused by the inherent armature reactance and the armature reaction at rated current. Referring to Fig. 444, the synchronous reactance is, according to some authorities, the ratio of field currents F_1/F_2, and according to others the ratio F_1/F_3. In most machines F_1 is only 5 to 15 per cent greater than F_2 and, therefore, the discrepancy is small. Fig. 444 shows the synchronous reactance to be 82.7 per cent by the first method and 75.2 per cent by the second method.

The sustained short-circuit current is obtained from the relation:

$$I_s = I\frac{F_s}{F_1}, \qquad \qquad (200)$$

where, I_s = sustained symmetrical r.m.s. short-circuit current per terminal in amperes;
I = rated current per terminal in amperes;
F_s = field current during short circuit;
F_1 = field current corresponding to rated current from the short-circuit curve.

The sustained short-circuit current of the alternator shown, if short-circuited at full load 90 per cent power factor, is 2040 amperes.

Transient reactance includes the leakage of the field and of the armature, and determines the total initial short-circuit current. On the occurrence of a short circuit the armature reaction quickly reduces the flux, and the initial current decreases in accordance with the decrement curve until the sustained

value is reached. The transient reactance of standard water-wheel-driven alternators varies from 15 per cent for high-speed designs to 30 per cent for low-speed designs.

Regulation may be obtained from the characteristic curves by reading the voltage from the saturation curve at any particular value of field current at which the regulation is desired. From Fig. 444 the regulation at field current corresponding to full load 100 per cent power factor is:

$$\text{Reg.} = \frac{E_1 - E}{E} = \frac{14{,}850 - 13{,}200}{13{,}200} = .125.$$

364. Ventilation.—The electrical losses of a generator all appear as heat, which must be carried off by direct radiation and by ventilation. With the large-diameter, slow-speed generator, the radiating surface is large in proportion to the heat generated and no particular attention need be given to ventilation in addition to the natural ventilation obtained by the open-type machine. With most of the modern generators, however, the heating presents a problem which, with the higher speeds, becomes very serious. Radiation is wholly inadequate on account of the comparatively small surface, and ventilation, which must be depended upon for most of the cooling, is very difficult on account of the compact construction. The moderate-sized generator is partially enclosed and provided with small fan blades mounted on the rotor. The larger-capacity generator is usually totally enclosed, and provided with fan blades and with definite intake and discharge air-duct connections. There are several systems of directing the cooling air through the machine, known as the radial, the axial, the circumferential, and other systems, but all essentially take air in at one or both ends, force it through ducts between windings and between sections of the core, and discharge it again at the periphery.

Ducts and dampers are often provided so that the air can be taken from out-of-doors or from the station and can be discharged out-of-doors or into the station, thereby providing a flexible means of maintaining comfortable station temperature during all seasons of the year. It is good practice to locate the air intake over the tail race at a safe distance above extreme high water. The air so obtained is generally cool and clean and contains some moisture, which renders it much more efficient as a cooling medium. When the fan action of the rotor itself is depended on to circulate the air, the ducts should be short and direct, and should have well-rounded corners. It is advisable to submit the duct design to the generator manufacturer before the final design of the rotor fans is completed. If the ducts are of excessively high resistance an external fan may be required. Roughly, 100 cu. ft. of air per minute will be required for each kilowatt of loss in the generator, and the velocity in the ducts should not exceed 1000 to 1500 ft. per minute. Exact data on the particular machine in question should be obtained from the manufacturer.

In case clean air cannot be readily obtained, consideration should be given to the use of air coolers and the closed system of air circulation as used in steam stations. Such a system is generally required at the substation for condensers and other rotating machines.

365. Temperature Detectors.—In order to determine the temperature of the interior of the generator armature while operating under load, temperature detectors are placed between adjacent coils or between coils and core, the leads of the detectors being brought out and connected to indicating instruments on the switchboard. Usually, at least six detectors are installed in a generator and all leads brought out to a terminal block on the frame. It is usually sufficient to connect three to the switchboard and reserve the other three for use in case any should become accidentally injured. The terminals on the generator frame should be provided with a protective device to prevent any abnormal voltage from the generator armature from being carried to the switchboard. There are two general types of detectors in common use: the resistance coil, and the thermo-couple.

The resistance coil consists of a flat coil of copper or alloy wire having a resistance, usually of 10 ohms, and a constant temperature coefficient of resistance. The instrument operates on the differential D'Arsonval principle. Three leads are used between the resistance coil and the instrument so as to eliminate the effect of the resistance of the leads.

The thermo-couple consists of two special dissimilar metals, such as constantan and copper, welded together to form an electrolytic couple. The two leads, one of constantan, the other of copper, are carried to the instrument and there joined together to form the cold joint. The instrument consists of a sensitive galvanometer and depends for its operation on the potential generated by virtue of the difference in temperature between the two junctions of the dissimilar metals.

366. Fire Protection.—Alternators are now almost universally provided with some sort of fire protection to reduce to a minimum the damage resulting from short circuits or internal fires. Water is probably the most common agent used. The manufacturer generally provides two coils of brass pipe so located and perforated as to direct a spray of water into the air path where it will be carried all over the windings. See Figs. 442 and 443. Care must be taken to prevent water from being delivered to the alternator when energized, and to this end the main valve should be sealed and tagged. To prevent any leakage from the main valve into the alternator there should be provided between the main valve and the alternator either a three-way valve or a small drip and a second valve.

Carbon tetrachloride, carbon dioxide, or some other inert gas or liquid is occasionally used in a piping system somewhat similar to the water piping system above described. The gases are, of course, most effective when a closed system of air circulation is used. The Lux system employs liquid carbon dioxide stored in steel cylinders under pressure and delivered to the generator through a specially designed valve which prevents freezing. In all cases it is important that the extinguishing agent be one that will not injure the insulation.

Smoke detectors have been tried on some generators with results indicating that they may come into general use. A representative sample of air is taken from the discharge duct of the generator and conducted to a part of the station where it can be conveniently seen by the operators. The Rich system

employs a small blower to conduct the air sample, and a lamp and lens to intensify the indication.

367. Installation.—The installation or erection of the generator on its foundation is generally performed by the manufacturer, except with very small machines. The foundation and bolts, however, are nearly always provided by the power-station builder. Whenever the erection is done by the builder, the manufacturer's instructions should be very carefully followed. Small generators can be shipped completely assembled, but larger machines must be assembled more or less completely on the foundations. It is common practice with the larger sizes to install the form-wound stator coils, and even to stack the laminated cores, as the machine is assembled on its foundation.

Foundations for water-wheel generators are commonly made of concrete and are usually an integral part of the power-house substructure. The foundation should season for two or three weeks before the machine base is installed. Bolts for anchoring the base should be cast into the concrete and accurately located by means of a template. It is good practice to provide each bolt with a large steel-plate washer to increase its holding strength, and to surround it with an iron pipe of sufficient size to give some lateral freedom. The foundation should be built up to within about 1 in. of the grade of the bottom of the machine base, to allow for grouting. After the base is set, the grouting should be given ample time to season thoroughly before the machine is placed upon the base. It is essential that the foundation extend under the entire base and positively prevent any deflection, since the manufacturer does not intend that the base be rigid enough to maintain proper alinement of the machine unless uniformly supported. The careful leveling and uniform grouting of the base are perhaps the most important details in the erection of the machine.

All machined surfaces and magnetic joints should be carefully cleaned before assembly. The bearings should have careful attention and should be amply supplied with clean oil. The air gap must be carefully equalized for all positions of the rotor. Before being energized, the machine should be carefully inspected and other precautions taken as described under Sec. 370.

368. Drying Out.—Generators that are assembled and tested before being shipped are usually dry and ready to run when they leave the factory. A machine of this type, however, as well as the coils of a machine that is assembled on its foundation, is likely to absorb moisture during transit, storage, and erection. Any such absorbed moisture is likely to cause an insulation failure and should be carefully driven out before the machine is brought up to voltage. A recommended method of drying is to run the machine at normal speed with the armature short-circuited beyond the ammeters so as to circulate enough current to warm the windings. The drying out is usually a part of the erection by the manufacturer, and if it is done by anyone else the manufacturer's instructions should be carefully followed. For generators not having embedded temperature detectors, it is usually recommended that the armature current, which is adjusted by field control, be maintained at about 110 per cent of normal. For generators with temperature coils, it is recommended that the temperature be adjusted to about 75° to 80° C. (total temperature,

not rise). In either case, it is advisable to check the temperature by thermometers placed about the stator.

The drying out should be continued until insulation measurements indicate conclusively that the machine is dry. The Standards of the A.I.E.E. specify that the insulation resistance of a clean, dry machine at operating temperature should not be less than:

$$\text{Ins. res. in megohms} = \frac{\text{Rated terminal voltage}}{\text{Rated kv-a.} + 1000} \quad . \quad . \quad . \quad (201)$$

The initial measurements, taken with the machine cold, are often misleading. Experience shows that the insulation resistance is rather high when the drying-out run is started, drops to a low value as the machine becomes warmed up, and then gradually rises to a more or less definite value which is maintained thereafter. More dependence should be placed on a uniform series of resistance measurements, even of moderate value, than on a fluctuating series, some of which may be quite high. Table LX gives some very condensed data from a typical drying-out run. Usually all readings are taken hourly, except the resistance readings, which are taken about every six hours. The test is usually continued for from two to six days. After the resistance readings have been fairly steady for at least a twenty-four hour period, it is generally considered that the machine is dry. Before putting the generator into service it is recommended that it be run for two or three minutes at a voltage of 10 to 25 per cent above normal.

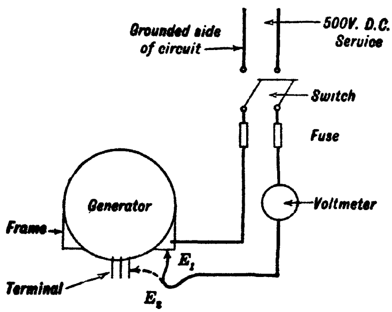


Fig. 446.—Diagram of Connections for Measuring Insulation Resistance by Voltmeter Method.

369. Measuring Insulation Resistance.—The insulation resistance of an alternator or other machine can be measured by a megger, or by the voltmeter method. The latter method is less subject to error, especially for the range of resistance values found in commercial machines, and is recommended in the Standards of the A.I.E.E. The connections are shown in Fig. 446. Two readings are required, E_1 with only the voltmeter in circuit, and E_2 with the voltmeter and the insulation in circuit. The insulation resistance is found from the following equation:

$$R_x = \frac{R_v(E_1 - E_2)}{E_2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (202)$$

where, R_x = resistance of insulation in ohms;
R_v = resistance of voltmeter in ohms;
E_1 = voltmeter reading with voltmeter only in circuit;
E_2 = voltmeter reading with voltmeter and insulation in circuit.

TABLE LX

TYPICAL DRYING OUT DATA OF 12,500-KV-A., 13,200-VOLT, 685-AMPERE 0.8-P.F., THREE-PHASE, 60-CYCLE, 1800-R.P.M. GENERATOR

These typical data are taken from an actual record of run which comprised a total of 97 sets of readings taken at one-hour intervals. Megger readings were taken about every eight hours.

Day	Hour	R.P.M.	Exciter Volts	Field Amps.	Armature Amps.	Temperature, Deg. C. Coil 1	Coil 2	Coil 3	Coil 4	Resist. Meg-ohms
Tues.....	10 a.m.	500	86	170	400	31	31	32	22	80
	8 p.m.	580	111	350	800	55	75	74	19	
Wed.....	1 a.m.	580	109	340	800	58	75	75	24	
	8 a.m.	600	106	330	780	61	76	76	26	13
	1 p.m.	520	109	310	740	61	75	75	24	
	8 p.m.	590	109	305	710	59	72	73	22	20
Thurs....	1 a.m.	600	109	315	720	60	75	75	22	
	8 a.m.	620	109	305	710	59	73	73	22	22
	1 p.m.	660	109	305	705	61	73	74	26	
	8 p.m.	C00	109	290	680	64	74	75	33	20
Fri......	1 a.m.	560	110	295	700	62	73	74	24	
	8 a.m.	620	106	300	700	61	74	75	26	22
	1 p.m.	600	106	280	660	63	74	75	34	
	8 p.m.	530	107	275	640	63	70	72	30	22
Sat......	1 a.m.	590	108	310	760	61	72	73	25	22
	10 a.m.	560	110	300	700	62	74	75	28	23

In making the test the following cautions should be observed:

(1) If either side of the direct-current circuit is grounded, the grounded side must be connected directly to the generator frame.

(2) The voltmeter must be connected in the ungrounded wire close to the source of supply.

(3) The wire between the voltmeter and the generator terminal must be very carefully supported, clear of all grounded objects, to prevent any possible leakage through its own insulation. It is advisable to suspend it free in the air.

(4) The voltmeter must have a high resistance, preferably about equal to the resistance being measured.

370. Starting.—Before starting a new alternator for the first time, it is advisable to give it a very thorough inspection to see that there are no loose pieces of iron or tools lying near that might be drawn into the rotating parts by the magnetic pull, that all air gaps are clear, that all electrical clearances are ample, that bearings are well supplied with clean oil, that oil rings are free to turn, and that brushes and brush holders are in good order. The external circuits should also be carefully inspected and cleared for operation.

The main and field breakers should be open. The machine should be started slowly by its prime mover and gradually brought up to speed. With the rheostat so adjusted that all its resistance is in the field circuit, the field breaker should be closed and the resistance then gradually cut out until the machine voltage comes up to normal. The main breaker may then be closed, connecting the alternator to the bus in case the bus is not already alive from any other source. In the majority of cases the alternator is not intended to operate on an isolated system, but must operate in parallel with other alternators or on a system already in operation. In this case the new alternator must be properly phased out and synchronized before being connected to the bus. (See Sec. 371.)

371. Phasing out and Synchronizing.—Before connecting an alternator to a bus or circuit which is already energized from another source, it is necessary, in order to prevent excessive and possibly disastrous current flow, that the alternating voltage across each phase of the alternator be equal at each instant to the alternating voltage across the corresponding phase of the bus. To meet this condition the alternator must fulfill the following conditions:

(1) Must have the same phase sequence as the bus.
(2) Must have the same voltage as the bus.
(3) Must have the same frequency as the bus.
(4) Must be in phase with the bus.

The first of the above conditions is determined by "phasing out" the alternator once for all when it is first installed. The last three are determined by "synchronizing" the alternator every time it is connected to the bus.

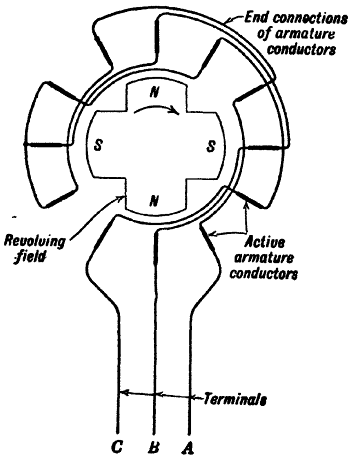


FIG. 447.—Diagrammatic Wiring of an Alternator Showing Phase Rotation.

Phasing out consists of testing and connecting the leads so that the phase sequence or sequence of positive peak voltages in the various phases of the alternator corresponds to that of the bus. Fig. 447 shows diagrammatically the winding of a three-phase, four-pole alternator with revolving field. The active conductors are shown radially as if flattened out into the plane of the page, although they are actually parallel to the shaft. It is evident that with a clockwise rotation of the field, as indicated by the arrow, the phase sequence is A–B–C. If the voltage on the bus were found to rotate C–B–A, which is opposite to the alternator, it would be necessary, before closing the breaker, either to reverse the direction of rotation of the field or to cross two of the leads between the alternator and the bus. Obviously, the latter method is always used, since the direction of mechanical rotation is determined by the prime mover.

If the phase sequence of the bus were accurately determined and definitely specified when the alternator was purchased, it would be possible for the manufacturer to build the machine to correspond by properly arranging the windings; but the crossing of the leads is generally such a simple matter that the manufacturer is practically always allowed to furnish his standard phase rotation, and no attempt is made to check up with existing conditions until the machine is about ready to run. It is essential, however, that some provision for crossing the leads be made in laying out the station wiring between the new alternator and the existing bus or circuit which is already energized from another source. The phase sequence of the existing bus should not be reversed, since this would reverse the direction of rotation of all synchronous and induction-type motors being fed by the bus; but often the cross in the wiring can be conveniently made at a transformer bank connecting the new alternator or new station to the existing system. Whenever a cross is made, care should be taken to make the necessary corresponding crosses in the instrument wiring.

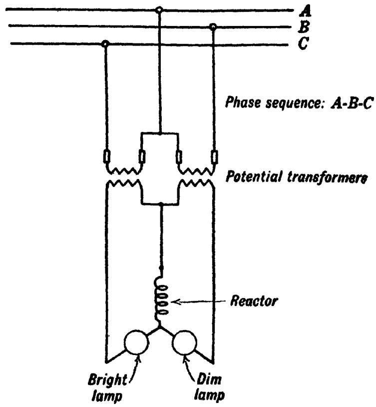


FIG. 448.—Diagram of Connections for Determining Phase Sequence.

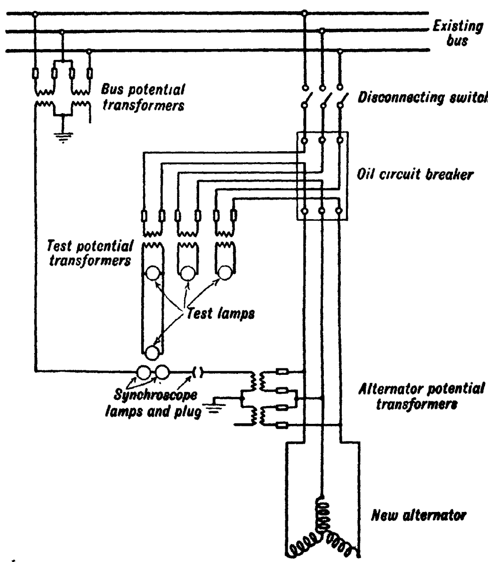


FIG. 449.—Diagram of Connections for Testing Phase Sequence and Synchronizing Connections of a New Alternator.

There are several ways of testing for actual phase sequence. Possibly the simplest is by the use of two lamps and a reactor, as shown in Fig. 448. The lamps should be of the same wattage and voltage, and for the reactor the potential coil of a watt-meter or watt-hour meter is satisfactory. When connected as shown, one lamp will burn " bright " while the other will burn

"dim," and, calling the reactor "black," the phase sequence is black-dim-bright or A–B–C. Another method is by the use of a phase-sequence meter as manufactured by The States Company, Hartford, Connecticut. It consists of a specially wound small transformer and a lamp, all contained in a small case. The lamp burns dimly if the phase rotation is clockwise and brightly if counter-clockwise.

When phasing out a new alternator, it is not necessary to determine the actual phase sequence of either the machine or the bus, but only to determine definitely that both are the same. A simple and accurate method of making this determination is illustrated in Fig. 449. The method has the advantages that little equipment and wiring are necessary, that no temporary main connections are necessary, and that the synchronizing connections are tested at the same time that the main connections are tested. The procedure is as follows:

(1) With the alternator disconnecting switches open, connect one potential transformer across each pole of the breaker and one lamp across each potential-transformer secondary. All this equipment should be located close to the breaker. An additional lamp located near the synchroscope on the switchboard should be connected across any one of the potential-transformer secondaries.

(2) With the breaker tied or blocked open, bring the alternator up to full speed and voltage; then close all three disconnecting switches. If the lamps at the breaker all blink together, the phase sequence of the alternator and of the bus are alike. If not, a cross must be made in any two main leads between the alternator and the bus, and the test repeated.

(3) After the phase sequence is found to be correct, insert the synchronizing plug. If the lamp located near the synchroscope and the lamps on the synchroscope all blink together and if all the lamps are dark when the pointer is in the synchronizing position, the secondary connections are correct. If not, a cross must be made in the secondary wiring between the alternator potential transformers and the synchroscope, and the test repeated.

(4) Remove all test equipment and wiring but do not disturb any of the alternator potential-transformer wiring or synchroscope connections, or any of the main leads. Then the machine may be synchronized in the usual manner.

Synchronizing consists of adjusting the voltage and frequency of the alternator to equal that of the bus, and closing the breaker at the instant the machine is in phase with the bus. The voltages of the alternator and of the bus are indicated by the switchboard voltmeters, and the former is adjusted by the alternator field rheostat. The frequency of the alternator as compared to the bus is indicated by the speed and direction of rotation of the synchroscope pointer and is adjusted by the governor control switch. The phase

relation between the alternator and the bus is indicated by the position of the synchroscope pointer, the vertical position generally indicating coincidence. A fraction of a second must be allowed for the operation of the breakers and a further allowance must be made for personal equation of the operator. Furthermore, it is easier on the machine to close slightly before perfect coincidence than after. For these reasons it is customary to adjust the speed of the machine so that the synchroscope pointer is revolving not faster than one revolution in two or three seconds, and then to close the breaker just before the pointer reaches the position of phase coincidence.

372. Parallel Operation.—Alternators that are to operate in parallel with each other and share the total load in proportion to their respective ratings must have the governors of their prime movers so adjusted that an increase in load causes a slight decrease in speed. The load on each alternator may be adjusted by changing its governor setting by means of the governor motor.

A change in the field current of an alternator operating in parallel with other alternators does not change its load, as with a direct-current generator, but does change its power factor. A decrease in the field current of an alternator operating in parallel with other alternators makes its power factor less lagging, and an increase in its field current makes its power factor more lagging.

373. Brakes.—Brakes are usually provided on a water-wheel-driven alternator to bring it to rest after being shut down, as without brakes slight leakage through the gates might cause the machine to rotate indefinitely. The brakes also constitute an additional safeguard against running away. On a horizontal machine the brake consists of a band or two shoes bearing against a flanged flywheel. On a vertical machine several brake shoes are usually applied to a machined surface on the bottom of the rotor. Wearing blocks are made of asbestos material. The braking force may be applied by magnets, oil, water, air or by purely mechanical means. Probably the most satisfactory medium is air, since it is usually available in the station, gives a smooth braking effect, due to its compressibility, and distributes the braking effect uniformly among the various shoes. With a vertical machine the same brake cylinders can be used with air for braking and with oil for jacking up the rotor.

The brakes are usually controlled manually from a point near the machine. They can be arranged to be applied automatically if required, and this is usually done in automatic stations.

The purchaser's specification for brakes should state the inertia of the alternator rotor and water-wheel runner, in WR^2; the leakage through the gates when closed, in per cent of full opening; the time in which the machine should be brought to rest; and the type and pressure of operating medium available.

374. Alternator Specification.—The following brief outline is intended as a guide to the principal items to be covered in a water-wheel-driven alternator specification:

General

Number wanted; reference to latest Standards of the A.I.E.E.; erection by purchaser or by contractor; material and workmanship; inspection; cooperation with other contractors.

Rating

Kilovolt-amperes; power factor (lagging); voltage; phase; amperes; frequency; speed; poles.

Type

Synchronous or induction; horizontal or vertical shaft; coupled, geared, or belted; kind of prime mover; for lighting, power, railway, or mixed load; revolving field or armature; sine voltage wave; for operation on transmission line.

Excitation

Excitation voltage; specification for direct-connected exciter if such exciter is required (see direct-current generator specification); range of voltage control by rheostat; rheostat form, whether panel or floor mounted; rheostat operation whether by hand, solenoid or motor; voltage of operating circuit; auxiliary switches on rheostat for indicating limits of travel and normal position; field discharge resistor.

Temperature Rise

Allowable temperature rise in degrees Centigrade of stator, rotor, collector rings, and bearings above an ambient temperature of 40° C. after "continuous" operation at rated kilovolt-amperes, power factor, voltage, and frequency; definition of "continuous," method of taking temperatures and hot-spot correction factors or reference to Standards of the A.I.E.E.; number and kind of temperature detectors, unless manufacturer's standard is acceptable; protection of temperature-detector leads against abnormal voltages.

Terminals

Whether one end only, or both ends of each phase of the armature winding shall be brought out to terminals; size of and approximate locations of armature terminals, field terminals and temperature-detector terminals.

Performance

Shall successfully endure sudden changes of load from no-load to full-load or *vice versa*, momentary short circuits or sustained grounds; shall successfully operate in parallel with each other and with existing alternators whose makes and serial numbers are given.

Mechanical Features

Description of proposed ventilating and air-conditioning system with approximate locations of inlet and outlet; size and type of coupling, gearing,

or pulley; type of base; number and type of bearings; weight to be carried by thrust bearing (if of vertical-shaft type); oil and water piping for bearings, water piping of brass; fire-protective features; direction of rotation; over-speed requirements; spare parts; finish (if different from manufacturer's standard); minimum flywheel effect; type of brake; WR^2 of entire rotating element; gate leakage; time allowed to stop rotation; type and pressure of brake-operating medium.

Data to be Submitted by Bidder

Guaranteed conventional efficiencies and regulations at various specified values of kilovolt-amperes and power factor and at rated voltage and speed; guaranteed field amperes and voltages at various specified values of kilovolt-amperes, power factor, and terminal voltage and at rated speed; resistance of armature and field windings; leakage, transient and synchronous reactances; type of insulation and insulation test voltage; exciter data (if exciter is included); quantity of air required per minute, and incoming and outgoing temperatures; weights of base, stator, rotor and heaviest piece; flywheel effect; outline drawing giving over-all dimensions.

Data to be Submitted by Contractor

Test values of guaranteed characteristics; detail drawings required for approval, for station design and for erection.

375. Flywheel Effect.—A change in the electrical load on the alternator causes a change in speed which, through the action of the governor or the water wheel, causes a corresponding change in the gate opening and driving effort. The change in driving effort always lags somewhat behind the change in electrical load and, as a result, the change in speed would be serious were it not for the flywheel effect of the revolving parts. Usually, the alternator rotor, together with the water-wheel rotor, has sufficient flywheel effect for satisfactory speed regulation; but with some designs it is necessary to provide the unit with a flywheel or with extra heavy rotors. Values of flywheel effect as given by manufacturers for standard alternators are given in Tables LVII and LVIII. The values are given in lb.-ft.^{2} The approximate per cent increase in speed with full load thrown off the alternator is:

$$s = \frac{90,000,000 \times K \times t}{WR^2 \times S^2}, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (203)$$

where, s = increase in speed in per cent of normal speed;
K = rating of alternator in kv-a. at 80 per cent power factor;
t = time in seconds for gate to close;
WR^2 = flywheel effect of entire revolving unit in lb.-ft.^{2};
S = normal speed of rotor in revolutions per minute.

376. Load Tests.—It is often necessary to provide an artificial load for testing a newly installed alternator or water wheel, because connections to the prospective system load are not completed or system-load conditions are not

suitable for tests. Various forms of energy-dissipating rheostats have been devised for this purpose. The rheostat forms a suitable load so far as the water wheel is concerned, but, owing to its non-adjustable unity power factor it is not so satisfactory for testing an alternator. Where a second generator is available a simple scheme consists of connecting the two generators together with one phase reversed, so that the second generator will be driven as a motor in reverse direction against the resistance of its water wheel. Various loads and power factors can be obtained by adjusting the gate opening and field rheostat of the motoring unit. This method is fully described by Mr. Robert Treat in an article entitled " A New Method of Artificially Loading Generators for Test," appearing in the *General Electric Review* for April, 1917, on page 333.

377. Alternator Charging Transmission Line.—When an alternator is connected to one end of a long transmission line which is open-circuited at its receiver end, the susceptance of the transmission line causes a low power-factor leading current to flow through the generator windings. The armature reaction within the generator, as a result of this leading current, strengthens the generator field flux and raises the voltage. This magnetizing action of an unloaded transmission line is often so great that a single generator connected to it may have its voltage built up to a dangerous value, even with very little or no excitation applied. It is, therefore, sometimes necessary to provide means of reversing the excitation to offset the magnetizing action or to avoid connecting a single generator to an unloaded line. If the situation is foreseen, the design of the generator can sometimes be modified so as to avoid excessive voltage from this cause. For further discussion of this subject, see " The Behavior of Alternating-current Generators when Charging a Transmission Line," by W. O. Morse in *General Electric Review* for February, 1920, page 109.

378. Excitation.—The excitation of an alternator is the power input required by the field winding to maintain the necessary intensity of magnetic flux. The effective flux is the resultant of the flux produced by the field current and that produced by the armature current. The latter is called armature reaction. With lagging power factor, the armature reaction opposes the field flux, while with leading power factor the armature reaction assists the field flux. Therefore, much greater field current is required with lagging power factor and heavy load than with leading power factor and light load. Since the power developed by the alternator is proportional to the product of the magnetic field strength and the speed with which the field flux cuts the armature conductors, it is evident that the slower the speed of rotation the greater must be the excitation for a given power rating. Approximate excitation requirements of standard water-wheel-driven alternators are given in Table LXI.

379. Excitation Systems.—Reliability is the prime requirement of an excitation system. Other important considerations are low first cost, economy of operation, simplicity, and convenience. The exciters should be of good design and liberal size; the method of drive should be reliable; the wiring should be short and simple and carefully installed; the method of control should be convenient and simple; and reserve capacity should be provided.

TABLE LXI

APPROXIMATE EXCITATION REQUIREMENTS OF WATER-WHEEL-DRIVEN GENERATORS

Rating, Kv-a.	Speed, R.P.M.	Excitation Requirements in Kw.	
		At 100% P.F.	At 80% P.F. Lag.
100	1200	2	3
	360	5	7
1,000	600	10	12
	150	19	25
5,000	600	30	38
	150	50	60
10,000	400	53	65
	150	75	95
20,000	360	85	110
	120	115	150
30,000	360	110	150
	120	140	190

The centralized excitation system, involving a bus fed by one or more exciters and feeding all the generator fields, has the advantage that a minimum number of large exciters may be used. A further possible advantage is that a battery may be floated on the bus as an emergency source. The principal objection to the centralized system is the possibility of a ground or other disturbance in one part affecting the entire system. The ideal centralized system would have three identical exciters, one of which could always be held in reserve.

The individual excitation system, involving an individual exciter associated with each generator, has the advantage that each generator with its exciter constitutes an independent unit not likely to be affected by faults originating in other units. A further advantage is that the exciter connections can usually be made shorter and simpler. A possible objection to the individual system, for a station having a large number of small generators, is that the exciters are comparatively small and inefficient. It is always advisable, with the individual excitation system, to provide a spare exciter connected to a bus extending to all generators for emergency use.

Motor drive is convenient, inexpensive, and fairly efficient. Induction motors are most simple, convenient, and reliable, although synchronous motors may be used if they are specially designed for stable operation during system disturbances. The source of energy to drive the motors must be uniform and reliable.

Water-wheel drive is relatively unreliable, expensive, and inefficient for small individual exciters, but is quite feasible for the larger exciters of a centralized system. At least one exciter must be driven by a prime mover for purposes of starting the station.

Dual drive, that is, by means of a motor on one end and a water wheel on the other, is often used to advantage. The water wheel can be used when starting the station, and either the water wheel or the motor for normal operation. In fact, it is customary to operate with both the motor and the water wheel connected to their respective sources of energy, and to adjust the governor so that practically all the energy is taken from one source or the other as desired. Then in case of failure of the driving source, the standby source will automatically assume the load.

Direct drive from the main unit is very efficient, reliable, convenient, and, except for very slow-speed units, inexpensive. Possible objections are that the generator voltage is particularly sensitive to speed fluctuations and that trouble with an exciter may cause the retirement of a large main unit. A spare exciter should be provided, except that when there are only two or three main units the individual exciters may be made large enough to supply two generators in case of emergency.

A storage battery provides a very reliable immediate standby source. Its high first cost, poor efficiency, and large space requirements, however, practically limit its usefulness to merely a momentary standby for use only while bringing a reserve exciter into service. For this reason, its greatest usefulness is obtained by floating it across the exciter bus, so that in case of failure of the exciter the generator excitation will not be interrupted, or at least by providing automatic transfer switches so that the time of interruption may be reduced to a minimum. An equally reliable standby can be obtained by operating a spare exciter continuously. Such an immediate standby, however, is seldom needed except in stations feeding out directly to important commercial customers, and batteries or continuously operating spare exciters are seldom used in hydroelectric stations.

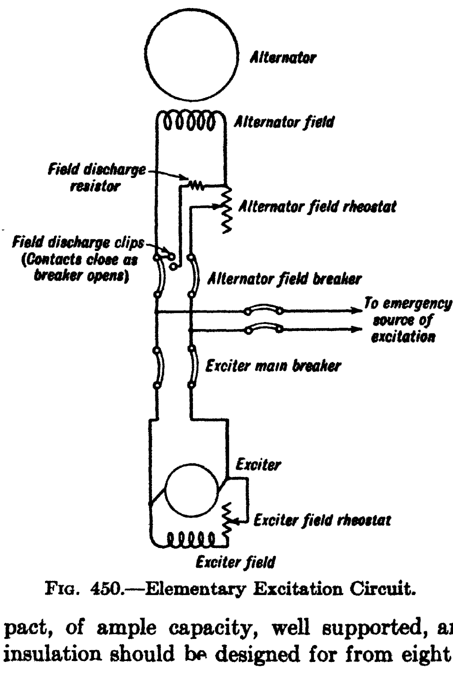


Fig. 450.—Elementary Excitation Circuit.

The voltage of the excitation system is generally 125 volts for small stations and 250 volts for large stations. For very large stations, higher voltages have been suggested in order to reduce losses, cable sizes, and duty on slip rings.

380. Excitation-system Wiring.—Wiring for the excitation system should be short, compact, of ample capacity, well supported, and carefully safe-guarded. The insulation should be designed for from eight to ten times the normal excita-

tion voltage, to take care of transient pressures due to generator short circuits. An elementary excitation circuit is shown in Fig. 450.

Breakers and rheostats are often hand-operated in small stations, but usually great advantage can be gained by making them electrically operated, since they can then be located most advantageously in the station to give short connections. The exciter main breaker is usually made automatic on reverse power, in order to disconnect the exciter in case of internal failure. The generator field breaker is non-automatic, but is often arranged to be tripped by generator relay action. It is not usually considered advisable to place overload protection on exciters as it is better to risk injury to the exciter than unnecessary operation of the automatic device. For instance, a sudden overload or short circuit in the generator circuit might cause sufficient rush of current in the exciter circuit to trip its breaker if it were automatic.

The excitation circuit must never be suddenly opened. The excessive voltage that would be built up by the sudden releasing of the energy stored in the inductive field winding might be sufficient to puncture the field insulation. To guard against this, the field breaker, which alone is used to break the circuit, is provided with discharge clips to be connected to the discharge resistor which is always furnished with the generator. As the breaker opens it connects the resistor across the field terminals and the energy is dissipated in the resistor. In some instances, interlocks are provided to prevent the opening of any breaker in the circuit until the field breaker is open, but to avoid complexity it is generally considered better to depend on the operator for proper sequence of operation.

381. Exciters.—An exciter is a direct-current generator designed especially to supply excitation for synchronous machines. The principal aim in the design for this purpose is to obtain stability under a wide range of voltage. For use with an automatic voltage regulator, the exciter should be so designed that its voltage will respond quickly to a change in its field current. For parallel operation with other exciters, it should have a drooping voltage-current characteristic at all operating voltages. Interpole windings are now practically standard for improving commutation. An exciter designed for use with an automatic voltage regulator is not necessarily as stable at lower voltages as one designed for manual voltage regulation, and, therefore, it is important to specify for which method of operation the exciter should be designed.

Approximate weights and dimensions of standard exciters are given in Table LXII.

The compound-wound exciter is usually preferred to the shunt-wound exciter where the alternating-current voltage regulation is accomplished manually, as the compound exciter requires less frequent adjustment to meet changing load conditions. Where automatic voltage regulators are used there is little choice between the compound exciter and the shunt exciter so far as the action of the regulator is concerned. Recent analysis indicates that the compound exciter is more stable in its operation during system disturbances than the shunt exciter. On the other hand, there are cases where, because of the magnetizing effect on the generator of a long transmission line, it is necessary to be able to reduce the exciter voltage to a very low value, and in this

TABLE LXII

APPROXIMATE WEIGHTS AND DIMENSIONS OF EXCITERS

Type	Rating, Kw.	Speed, R.P.M.	Weight in Pounds		Dimensions in Inches		
			Shipping	Net	Length	Width	Height
Coupled	25	150	3,700	3,100	31	40	46
(Horizontal)	50	150	8,700	7,300	43	56	57
	100	150	14,500	12,000	57	67	65
	150	120	23,000	20,000	61	80	76
	200	120	31,000	26,000	66	90	84
Belted	25	720	1,720	1,450	45	30	35
(Including	50	720	3,000	2,550	55	36	52
pulley and	100	720	4,200	3,500	64	40	46
base)	150	720	9,500	8,000	108	57	51
	200	720	12,500	10,500	112	57	51
Motor-driven	25	1750	2,500	2,080	72	25	32
(Including	50	1150	6,400	5,300	96	40	45
motor)	100	1150	8,500	7,000	94	43	52
	150	1160	12,500	10,500	111	47	58
	200	1200	15,500	12,400	118	50	50
	250	1200	18,200	15,000	137	47	66
	300	1200	19,000	15,600	138	49	70
Water-wheel-	25	720	1,730	1,450	45	30	35
driven	50	720	3,100	2,550	55	36	52
(Horizontal,	100	720	4,250	3,500	64	40	46
including	150	720	8,200	6,900	74	47	49
bearings, base	200	720	11,000	9,200	90	51	52
and shaft)	250	600	15,000	12,500	90	56	56
	300	600	15,000	12,500	90	56	56
Motor and	25	900	4,200	3,500	82	34	42
water-wheel-	50	720	7,300	6,075	98	40	45
driven	100	720	11,500	9,500	110	44	57
(Including	150	720	24,000	20,000	140	56	57
motor)	200	720	28,800	24,000	172	64	65
	250	600	34,200	28,500	184	64	65
	300	600	34,200	28,500	184	64	65

Data from General Electric Company and Westinghouse Electric & Manufacturing Company.

case the shunt exciter possesses some advantages. Compound exciters, if operated in parallel, require equalizer connections which complicate the station wiring.

382. Direct-current Generator Specification.—With slight modification, this can be used as an exciter specification.

General

Number wanted; reference to latest Standards of the A.I.E.E.; erection by purchaser or by contractor; material and workmanship; inspection; cooperation with other contractors.

Rating

Kilowatts; volts: amperes· speed; shunt, compound or interpole field winding.

Type

Two or three-wire; horizontal or vertical-shaft; coupled, geared, or belted; kind of prime mover; for lighting, power, railway, excitation, or mixed load; self or separately excited.

Excitation

Excitation voltage if separately excited; specification for direct-connected exciter if such exciter is required; range of voltage control by rheostat; rheostat form, whether panel or floor mounted; rheostat operation, whether by hand, solenoid, or motor; voltage of operating circuit; auxiliary switches on rheostat for indicating limits of travel and normal position; field discharge resistor.

Temperature Rise

Allowable temperature rise in degrees centigrade of armature, field, commutator, and bearings above an ambient temperature of 40° C. after " continuous " operation at rated kilowatts, voltage, and speed; also, if desired, after " one hour " or " two hour " operation at 125 per cent or 150 per cent rated kilowatts, and at rated voltage and speed; definition of " continuous " ".one hour " and " two hour " method of taking temperatures and hot-spot correction factors for reference to Standards of the A.I.E.E.; number and kind of temperature detectors unless manufacturer's standard is acceptable; protection of temperature-detector leads against abnormal voltages.

Terminals

Sizes and approximate locations of positive, negative, neutral (if three-wire), field and temperature-detector terminals; location of switch for neutral lead (if required); location of conduit terminal box (if required).

Three-wire Features (if Required)

Amount of unbalance required; shall safely withstand all the load of either side of system being thrown on or off; allowable amount of unbalanced voltage corresponding to maximum unbalanced load; series windings equally divided between positive and negative sides; method of obtaining neutral, whether by internal compensator or external compensator.

Mechanical Features

Description of proposed ventilating and air conditioning system with approximate locations of air inlet and outlet; size and type of coupling, gearing or pulley; type of base; number and type of bearings; weight to be carried by thrust bearing (if of vertical-shaft type); oil and water piping for bearings, water piping of brass; fire-protective features; direction of rotation; over-

speed requirements; spare parts; finish (if different from manufacturer's standard); minimum flywheel effect.

Starting Requirements (if Required to Start as a Motor)

Starting rheostat of sufficient size to start the set a specified number of times per hour; description of any special arrangement for starting.

Regulation

Allowable voltage regulation, including that due to speed regulation of prime mover; adjustable shunts for compound and interpole windings.

Performance

Shall successfully endure, without serious flashing or injury and without necessity of shifting the brushes, sudden changes of load from no-load to full-load or *vice versa*, momentary short circuits or sustained grounds; shall successfully operate in parallel with each other and with existing generators whose makes and serial numbers are given.

Data to be Submitted by Bidder

Guaranteed conventional efficiencies at various specified values of kilowatts and at rated voltage and speed; guaranteed field amperes and voltages at rated load, hot, and at no-load, cold; number of poles; resistances of armature and field windings; type of insulation and insulation test voltage; exciter data (if exciter is included); quantity of air required per minute and incoming and outgoing temperatures; weights of base, field frame, armature and heaviest piece; flywheel effect; outline drawing giving over-all dimensions.

Data to be Submitted by Contractor

Test values of guaranteed characteristics; detail drawings required for approval, for station design and for erection.

383. Rheostats.—In order to allow a reasonable margin of safety for variations in design and materials, it is customary to wind the alternator field so that about 90 per cent of the exciter voltage will be required for rated load and power factor. A rheostat is then provided in the alternator field circuit to control the excitation as required. The rheostat is usually designed with sufficient resistance to reduce the alternator voltage to about 85 per cent of normal with no-load, full-speed, alternator cold and full exciter voltage. On this basis an alternator rheostat of usual design with cast-iron resistor grids will weigh approximately 25 lb. per kilowatt of excitation.

384. Transformers.—A transformer is defined as a device for transferring electrical energy from one alternating-current circuit to another alternating-current circuit. The essential features are (1) a primary winding which receives energy from the supply circuit, (2) a secondary winding which delivers

energy to the receiving circuit, and (3) an iron core common to both windings. The windings are so arranged on the core that alternating current in the primary from the supply circuit induces a magnetic flux in the core, which in turn induces an alternating current in the secondary.

The high-voltage winding is the one having the greater number of turns, and the low-voltage winding is the one having the lesser number of turns. The ratio, unless otherwise specified, is always understood to mean the turn ratio. If the transformer were ideal, that is, without losses, the voltage ratio would be equal to the turn ratio and the current ratio would be the reciprocal of the turn ratio at all loads. Due to the transformer losses, however, the primary voltage and current are higher than the values obtained by applying the turn ratio to the secondary voltage and current.

A constant-potential transformer, that is, one that at all loads has a constant voltage ratio, except as slightly modified by losses, is the type that is commonly used for transmission and distribution of power. Sizes below 200 kv-a. are often spoken of as distribution transformers, and those of larger size as power transformers.

A single-phase transformer, as distinguished from a polyphase transformer, has a single primary winding and a single secondary winding. It can be used by itself in a single-phase system or banked with other similar transformers in a polyphase system. The three-phase transformer is the only polyphase type in common use. It has three primary and three secondary windings arranged on a common core, and is used by itself in a three-phase system. Advantages of the single-phase transformer are as follows: (1) banks of larger size can be shipped; (2) a defective transformer can be replaced more easily; (3) a single unit will suffice as a spare for several banks; and (4) by various groupings many different kinds of transformations may be obtained. Advantages of the three-phase transformer are as follows: (1) the external connections are simpler; (2) less floor space is required; (3) the total weight is less; and (4) the first cost is less. One manufacturer shows a saving of 50 per cent in both floor space and weight for certain sizes of three-phase transformers as compared with three single-phase transformers of the same total capacity.

A three-phase delta-connected shell-type transformer with one phase damaged may be operated in open delta at 58 per cent capacity by short-circuiting both high-voltage and low-voltage windings of the damaged phase. If the transformer is of the core type, it cannot be operated in open delta unless the high- and low-voltage windings of the damaged phase can be open-circuited.

Outdoor and indoor type transformers have in the past differed with respect to the types of bushings used. With the larger sizes and higher voltages, however, it has become customary to make all power transformers suitable for either outdoor or indoor service.

385. Construction.—Transformer cores are built up of thin sheets or laminations of special transformer steel having high magnetic permeability. To prevent eddy-current losses, the laminations are varnished before assembly, or otherwise insulated from each other and from the bolts which bind them together. The core is built up into one of two general types, the core type and

the shell type, Fig. 451. Each type has particular advantages as to winding space, insulation, and bracing, and the choice depends upon the capacity, voltage, and frequency.

The windings are of two general types: cylindrical windings, assembled concentrically; and discoidal windings, assembled interleaved, Fig. 452. The choice depends upon the capacity, voltage and frequency. Often the two types are combined in various ways. The relative grouping of the primary and secondary coils determines to a large extent the reactance of the transformer. It is essential that the windings be firmly braced to resist severe mechanical stresses without impeding the free circulation of oil about the conductors.

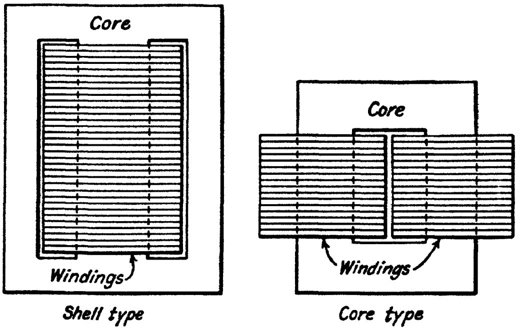


Fig. 451.—Two General Types of Transformers.

Insulation is one of the most important features of the transformer construction. The individual conductors of a coil are usually wrapped with cotton or cambric, sometimes paper, and the coil is taped with cotton or cambric tape. The coils are then vacuum-treated and impregnated with insulating compound. In assembly, the coils are insulated from each other and from the core by sheets of fuller board or similar material. For air-blast transformers the coils are treated with waterproof compound and well separated, particularly at the ends of the coils where potential stresses are greatest. In the oil-insulated transformer the core and coils are immersed in oil which serves as both an insulating and a cooling medium. For high voltages some manufacturers provide a potential shield at the high-voltage end of the coil, which helps to distribute the abnormal potentials of line disturbances over a number of the adjacent coils, the action being much the same as that of a shield on the end of a string of suspension insulators.

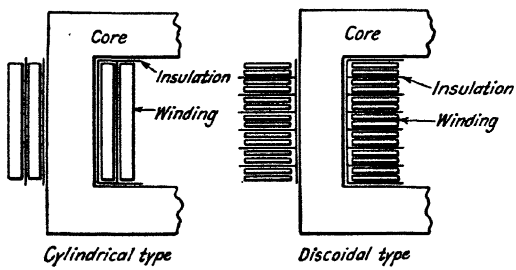


Fig. 452.—Two General Types of Transformer Windings.

Bushings for bringing the leads out through the case are of three general types: the solid type, the condenser type, and the oil-filled type. The solid bushing, used for voltages up to about 60,000, consists of a tube of porcelain

or other insulating material, usually with petticoats on the part above the transformer case. The condenser type, which is suitable for the higher voltages, consists of a round conductor wrapped by alternate layers of treated paper and tin foil, the successive layers being of increasing diameter and decreasing length toward the outside of the bushing, thus distributing the potential stress evenly over the entire length and thickness of the bushing. The oil-filled bushing, which is also adaptable to the higher voltages, consists of a tube, made up of a number of concentric pieces of porcelain, filled with oil and containing a conducting rod or tube through its center. At voltages of over 100,000, care must be taken to prevent corona either inside or outside the case. The bushing should have a puncture voltage higher than the flash-over and should have a flash-over time lag exceeding that of the lightning arrester used to protect it. It is found that the flash-over voltage at lightning frequency is practically the same under wet or dry conditions and is in the neighborhood of twice the normal frequency dry flash-over.

The case or tank is usually built of boiler plate riveted and welded oil-tight. For the self-cooled transformer the case is fluted and sometimes also provided with external radiators. For water-cooling the case is smooth. Cooling coils should be arranged to drain themselves when the water supply is shut off, so as to prevent freezing. They should preferably be of such shape that the core and windings can be lifted out of the tank without disturbing the coils. The case should be provided with wheels or castors, lifting and pulling lugs, thermometer, oil-drain valve, sampling valve, filtering connections, and oil gage.

Conservation of the oil to exclude moisture and oxygen is of great importance. Moisture very greatly reduces the insulating properties of the oil, and the presence of oxygen, together with high operating temperatures, causes the oil to sludge. With changes of load, the resulting expansion and contraction causes breathing, and various methods have been developed for eliminating moisture and oxygen from the air breathed into the case. The chloride of lime breather has been used extensively. If recharged frequently, it is quite effective. The conservator or expansion-tank transformer, now extensively used, is arranged to breathe through a small conservator or expansion tank located above the main tank and connected to it by a small pipe. The main tank is completely filled with oil, the only oil surface exposed to air being that in the small tank. Circulation between the main tank and the small tank is so restricted that the oil in the small tank remains cool and, therefore, sludging is prevented. The inertaire transformer is arranged to breathe through a deoxidizing compound, which not only relieves the incoming air of its oxygen, but also absorbs the oxygen from the air left in the transformer at the start. As a result, an atmosphere of nitrogen is maintained above the oil, and sludging is avoided.

Relief valves or diaphragms are usually provided to open or blow out in case of excessive internal pressure due to flashing or burning inside the case.

386. Rating.—The rated output of a transformer is the product of the volts and amperes at its secondary terminals when delivering its maximum continuous load, and is expressed in kilovolt-amperes. The maximum continuous

output is limited by the heating. The Standards of the A.I.E.E. specify the limiting temperatures for various classes of insulation and the methods of measuring the temperatures.

The rated primary voltage, as defined in the Standards of the A.I.E.E., is the rated secondary voltage multiplied by the turn ratio. It follows that when the transformer is carrying load at rated secondary voltage, the primary voltage is higher than rated by an amount equal to the regulation. The test voltage specified in the Standards of the A.I.E.E. is twice the normal voltage of the circuit to which the transformer is connected plus 1000 volts at not less than rated frequency and for a period of sixty seconds.

Single-phase transformers for star connection to three-phase circuits must, in complicance with A.I.E.E. rules, be tested to 3.46 times transformer voltage plus 1000 volts, that is, twice line voltage plus 1000 volts. For very high voltage transformers, the amount of insulation required to meet this test is very expensive and in cases where the neutral of the system is to be solidly grounded at all times the test voltage is sometimes specified as 2.73 times transformer voltage plus 1000 volts, that is, transformer voltage plus line voltage plus 1000 volts.

387. Taps and Internal Connections.—Taps are usually provided to counteract line drop and transformer regulation. They are convenient for the operation of the system, but from the manufacturer's standpoint are somewhat hazardous and expensive, especially in high voltage transformers and, therefore, should be specified only when actually required. Full-capacity taps are generally desirable, and their use requires that the windings have current capacity to supply rated kv-a. at lowest tap voltage. Where reduced-capacity taps can be used, the cost of the transformer can be somewhat reduced. A tap-changing switch, whereby the taps can be selected by means of a handle outside of the transformer case, is quite essential where system operation requires frequent changing of taps. The ordinary tap-changing switch cannot be operated with the transformer under load.

Both primary and secondary windings are often arranged in groups which can be connected either in series or in parallel to give a selection of voltages. This feature is sometimes desirable for operating a system at half voltage initially, before the load is fully developed. The name plate should show clearly all taps and internal connections for which the transformer is designed.

388. Parallel Operation, Polarity.—Parallel operation of two or more transformers, or two or more banks of transformers, with distribution of the total load between them in proportion to their respective ratings, requires that they have (1) the same polarity, (2) equal voltages and turn ratios, and (3) equal per cent impedances. If the ratios of the transformers are not equal, one may be corrected by the use of an auto-transformer connected in series. If the impedances are not equal, one may be corrected by the use of an external impedance connected in series. Such devices are only used, however, in cases of emergency. The manufacturer, if provided with data on the existing transformers, can nearly always design the new transformers to operate satisfactorily in parallel with the old ones.

Polarity of a transformer refers to the relation of its primary instanta-

neous voltage to its secondary instantaneous voltage, and this, of course, involves the location of its leads. A subtractive polarity transformer is one whose windings and leads are so arranged that, by connecting a high-voltage lead to the adjacent low-voltage lead and applying voltage to either winding, the voltage between the other two leads will be less than the voltage across the high-voltage winding. An additive polarity transformer is one whose windings and leads are so arranged that under the above test the voltage between the other two leads will be more than the voltage across the high-voltage winding. The two kinds of transformers are shown diagrammatically in Fig. 453, and the method of test for polarity is shown in Fig. 454. For purposes of standardization, all single-phase transformers of over 200 kv-a. rating, and also those of 200 kv-a. or less if of over 7500 volts, are made subtractive polarity and are so marked on their name plates. Recommended markings of leads are given in the Standards of the A.I.E.E.

389. Resistance, Reactance and Impedance.—Resistance, reactance, and impedance of a transformer may be expressed in ohms, or as internal voltage

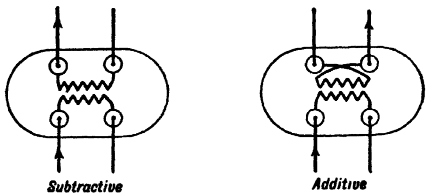


FIG. 453.

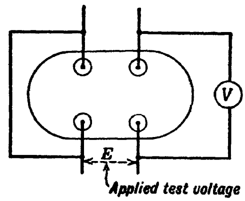


FIG. 454.

FIG. 453.—Transformer Polarity. Arrows indicate instantaneous current flow.

FIG. 454.—Test for Transformer Polarity. *V* less than *E* indicates subtractive polarity. *V* greater than *E* indicates additive polarity.

drops, in percentages of rated secondary voltage, caused by rated current at unity power factor. The relation between the percentage drop and the ohmic drop is:

$$\frac{\text{Per cent drop}}{100 \times \text{ohms}} = \frac{\text{Rated current}}{\text{Rated voltage}} = \frac{\text{Rated volt-amperes}}{(\text{Rated voltage})^2} \quad . \quad . \quad (204)$$

If the resistances of the primary and secondary windings are given separately, the primary resistance may be converted to equivalent secondary resistance by multiplying it by the square of the reciprocal of the turn ratio. The equivalent resistance of the primary can then be added to the secondary resistance to find the total transformer resistance in terms of secondary ohms. By a similar process the secondary resistance can be converted to equivalent primary resistance and added to the primary resistance to give the total transformer resistance in terms of primary ohms. For example, a transformer having a turn ratio of $\frac{1}{6}$, a primary resistance of 2 ohms, and a secondary resistance of 70 ohms, has a total resistance expressed in terms of secondary ohms of:

$$70 + 2 \times \tfrac{36}{1} = 142 \text{ ohms},$$

and expressed in terms of primary ohms of:

$$2 + 70 \times \frac{1}{36} = 3.94 \text{ ohms}.$$

Reactance and impedance may be converted similarly, but it is seldom necessary to consider the reactance or impedance of either winding separately.

Low resistance is desirable in order to reduce losses, and low reactance is desirable in order to give good regulation. In connection with large power systems, however, it is usually necessary to build transformers with comparatively high reactance in order to reduce short-circuit currents.

390. Regulation.—The regulation of a transformer is the difference between the no-load secondary voltage and the rated-load secondary voltage expressed in per cent of rated secondary voltage, the primary voltage being constant at such value as to give rated secondary voltage at rated load. The regulation varies with the power factor of the load and, therefore, the power factor at which the regulation is given must always be stated. The regulation may be obtained by measuring the change in voltage with change in load, or by measuring the resistance and reactance and computing the regulation from the equation:

$$\text{Per cent regulation} = q_r \cos\theta + q_x \sin\theta + \frac{(q_x \cos\theta - q_r \sin\theta)^2}{200}, \quad . \quad . \quad (205)$$

where q_r = per cent resistance drop;
q_x = per cent reactance drop;
$\cos\theta$ = power factor;
$\sin\theta$ = reactive factor.

391. Efficiency.—The efficiency of a transformer is the ratio of the kilowatt output to the kilowatt input. The losses are classified as no-load losses, or those that continue as long as the transformer is energized regardless of whether it is delivering any load, and load losses, which are dependent on the load delivered. The Standards of the A.I.E.E. specify that the conventional efficiency of a transformer shall be determined as follows:

No-load losses shall include the core loss, the I^2R loss due to the exciting current and the dielectric loss in the insulation at 75° C. They shall be measured with open secondary circuit at rated frequency, and with an applied primary voltage, giving the rated secondary voltage plus the IR drop which occurs in the secondary under rated load.

Load losses shall include I^2R losses, and stray-load losses due to eddy currents caused by fluxes varying with load at 75° C. They shall be measured by wattmeter by applying a primary voltage, at rated frequency, sufficient to produce rated load current in the windings, with the secondary windings short circuited.

With the losses given, the efficiency may be computed from the equation:

$$E = \frac{100P}{P + L} = 100 - \frac{100L}{P + L} = 100 - \frac{100\,(I + C)}{P + (I + C)}, \quad . \quad . \quad (206)$$

where E = efficiency in per cent;
P = delivered power in kilowatts;

L = losses in kilowatts;
I = no-load losses (commonly called iron losses) in kilowatts;
C = load losses (commonly called copper losses) in kilowatts.

It should be noted that with a fixed value of kv-a., P is directly proportional to the power factor, I is independent of load or power factor, and C is proportional to the load but independent of the power factor. It is necessary, therefore, when giving the efficiency of a transformer, to state the load and power factor at which it applies. It is usual for the manufacturer to submit with his quotation the efficiencies at several loads and at unity power factor. The full-load, 100 per cent power-factor efficiency of the modern 60-cycle power transformer, varies from 96 per cent to 99 per cent, depending on size and voltage. The 25-cycle transformer is slightly less efficient.

By properly proportioning the iron and copper losses, the manufacturer can usually obtain the highest efficiency at 50, 75, or 100 per cent load, as desired, to give the best all-day efficiency. For instance, a transformer which is to carry full load all the time should have its best efficiency at 100 per cent load, which means that the iron losses and the copper losses should be about equal at full load, while a transformer which is to carry only partial load many hours of the day should have its best efficiency at less than 100 per cent load, which means that the iron losses and the copper losses should be about equal at less than full load.

392. Cooling.—The losses of the transformer appear as heat, which must be carried away as fast as formed so as to prevent excessive temperature rise; and the method of cooling classifies the transformer in one of four general types which are, in order of decreasing importance: self-cooled, water-cooled, forced oil-cooled and air-blast.

The self-cooled transformer has the core and windings immersed in oil, and the case is either fluted or provided with radiators so that the oil is cooled by natural radiation. The oil circulates by convection upward through the hot windings and downward through the flutes or radiators. This type can be built in practically any commercial capacity and voltage. With the very large capacities, however, the radiators become larger than the transformer cases themselves and, therefore, become bulky and expensive. The great advantage of this type is its simplicity.

The water-cooled transformer is also oil-insulated but the tank is plain. Around the inside of the tank at the top, where the oil is hottest, a cooling coil is placed, through which cold water is forced. Cooling water at a hydroelectric generating station is usually available in such quantities that the discharged water can economically be rejected. At the substation, however, water is usually too expensive to waste and must be cooled in a cooling tower or spray pond and recirculated. It is desirable to use a low water pressure to prevent the possibility of water leaking into the oil, and for this reason it is customary to discharge the water from the transformer at atmospheric pressure.

The forced oil-cooled transformer is similar to the water-cooled, except that it has no water coils. The oil is circulated by pumps through external

radiators which are cooled by air, or through surface coolers which are cooled by water. Oil circulation should be upward through the transformer and downward through the cooler. Water circulation should be upward through the cooler. It is important that the oil pressure be maintained higher than the water pressure to prevent water leaking into the oil. Purity of the water is not as important as with the water-cooled transformer, for the cooler can be made in sections and part at a time can be cleaned and inspected. At the generating station it is sometimes possible to place the oil-cooling coils or radiators in the tail race.

The amount of cooling water required for a water-cooled transformer or for a forced oil-cooled transformer, and the amount of oil circulation required for a forced oil-cooled transformer are given by the following equations, which do not take into account any radiation. In the case of forced oil-cooled transformers the actual amount of water required may in some instances be found to be 10 or 20 per cent less, owing to radiation from the oil piping and coolers.

$$Q_w = \frac{3.8P}{T_w}, \qquad (207)$$

$$Q_0 = \frac{4.5P}{T_0 S_0}, \qquad (208)$$

where Q_w = gallons per minute of cooling water required for water-cooled or forced oil-cooled transformers;

Q_0 = gallons per minute of oil circulated between transformer and cooler for forced oil-cooled transformers;

P = transformer loss in kilowatts;

T_w = temperature rise in degrees Centigrade between ingoing and outgoing water;

T_0 = temperature rise in degrees Centigrade between cool oil and hot oil, usually equal to 10 to 15;

S_0 = specific heat of oil, equal to 0.40 to 0.50.

393. Oil.—Oil for transformers is obtained from crude petroleum by fractional distillation. Desirable qualities are (1) high resistivity and dielectric strength, (2) low viscosity, (3) high flash and burning points, (4) high thermal conductivity and specific heat, (5) chemical neutrality toward metals and insulating materials, and (6) chemical stability at high temperatures. Characteristics of commercial transformer oil are as follows:

Flash point, degrees Centigrade	130 to 140
Burning point, degrees Centigrade	140 to 150
Freezing point, degrees Centigrade	0 to −10
Specific heat	0.39 to 0.51
Weight, pounds per gallon	6.9 to 7.1
Viscosity at 40° C., Saybolt test, seconds	40 to 50
Dielectric strength, between 1 in. disks 0.1 in. apart, volts	20,000 to 30,000

Moisture, even in very minute quantities, greatly reduces the dielectric strength of oil. Before being put into service, the oil should be tested and should not break down at less than 22,000 volts. This requires that it shall contain less than about 0.001 per cent moisture by weight. It should be sampled about once a month while in service, and when the breakdown voltage falls to 17,000 volts it should be dried again.

There are two common methods of drying and purifying oil: (1) by passing it through a centrifuge, and (2) by forcing it through dry blotting paper. The former method effectively removes the heavy foreign matter and some of the water. In order to use the centrifuge to best advantage, the oil must be warmed and, unfortunately, its tendency to hold moisture increases enormously with a moderate rise in temperature. The filter-press method removes all suspended matter, and, if proper attention is given to drying the blotting papers, thoroughly and frequently, in an electric oven, this method removes practically all traces of moisture. A combination of the two methods is very effective.

394. Transformer Specification.—The following brief outline is intended as a guide to the principal items to be covered in a power transformer specification:

General

Number wanted; reference to latest Standards of the A.I.E.E.; erection by purchaser or by contractor; material and workmanship; inspection.

Rating

Kilovolt-amperes; phase; cycles; volts of high-voltage winding; volts of low-voltage winding; kind of cooling; outdoor or indoor.

Connections and Taps

Number of transformers to be used in a bank; voltage, phase and connection of high-voltage side; voltage, phase and connection of low-voltage side; high-voltage, coil connections in series or parallel for multiple voltages; low-voltage coil connections in series or parallel for multiple voltages; high voltage and low voltage for which coils are to be connected when transformer is shipped; voltages and kilovolt-ampere capacities of high-voltage taps; voltages and kilovolt-ampere capacities of low-voltage taps; whether tap-changing switch is required for changing taps from outside the case.

Temperature Rise

Allowable temperature rise in degrees Centigrade above an ambient temperature of 40° C. for air or 25° C. for water after " continuous " operation at kilovolt-amperes, voltage, and frequency; definition of " continuous," method of taking temperatures and hot-spot correction factor or reference to Standards of the A.I.E.E.

Performance

Shall successfully endure momentary short circuits or sustained grounds with sustained primary volts; shall successfully operate in parallel with each other and with existing transformers whose makes and serial numbers are given.

Insulation Test

Test voltage at rated frequency from high-voltage winding to low-voltage winding and core; test voltage at rated frequency from low-voltage winding to core; excitation at twice normal voltage at suitable frequency; duration of each test in seconds.

Cooling

Type of cooling, whether by radiation, water, forced oil or other means; whether cooling equipment is to be included.

Mechanical Features

Type of wheels; eye-bolts and lugs for lifting the complete transformer, for jacking it up from the rails, for pulling it horizontally, for removing a bushing, for removing the cover, and for removing the cover and core and windings as a unit; material of cooling coils and arrangement so as to drain by gravity and so that core and windings can be removed without disturbing them; manifolds, valves, and flow indicators for cooling coils; pressure of cooling-water supply and test pressure of cooling coils; oil-drain valve, sediment valve, sampler valve, filter-press connections, and oil gage; breathers, conservators, expansion tanks, or other devices; temperature indicator with alarm contacts; temperature detectors; sketch showing desired locations of terminals, axles, wheels, and tank fittings.

Rating Plate

Should contain complete rating, gallons and kind of oil, gallons per minute and pressure of water, additive or subtractive polarity, and diagram of connections and taps.

Data to be Submitted by Bidder

Guaranteed conventional efficiencies and regulations at various specified values of kilovolt-amperes and power factor and at rated voltage and frequency; reactance, resistance, and impedance volts in per cent of rated volts at rated load, frequency, and voltage, and at unity power factor; no-load losses at rated volts and frequency, including iron, dielectric, and exciting-current losses; copper losses at rated load; exciting-current amperes and power factor; quantity and grade of oil; flow of water required for cooling in gallons per minute, and drop in pressure through coils in pounds per square inch, at rated load and at half load, if water-cooled; flow of oil required for cooling in gallons per minute, drop in pressure through transformer in pounds per square inch and temperature of ingoing and outgoing oil, at rated load and half load, if forced oil-cooled; correction factor for obtaining temperature at instant of

shut-down; details of construction; approximate shipping and net weights of cover, core and windings, of case, of oil and of complete transformer; approximate dimensions of length, width, height less bushings, height with bushings; total distance from bottom of wheels to top of bushings when lifting cover, core, and windings over top of tank.

Data to be Submitted by Contractor

Test values of guaranteed characteristics; detail drawings required for approval, for design, and for erection.

395. Connections.—Of the large number of possible connections and groupings of transformers, the most commonly used are shown in Table LXIII.

396. Installation.—Installing transformers outdoors saves building space, obtains good ventilation, and reduces fire hazard. When installed indoors, they should be surrounded by substantial fire-resisting walls and, unless of the water-cooled type, should be provided with adequate ventilation to carry off the losses. It is desirable to leave clear space all around each transformer, for inspection, reading of gages, and operation of valves. The tanks should be connected to suitable oil-drain pipes, and it is also convenient to provide a tank into which the oil from the transformers can drain by gravity.

TABLE LXIII

Standard Transformer Connections

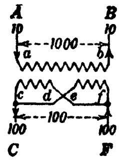
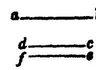
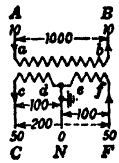
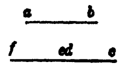
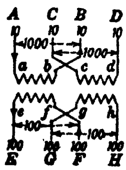
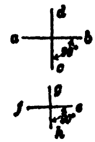
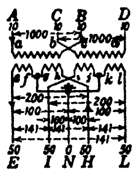
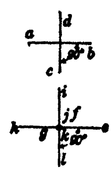
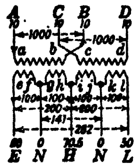
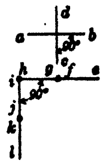
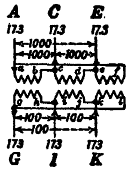
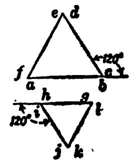
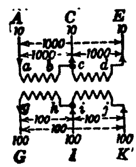
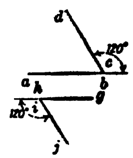
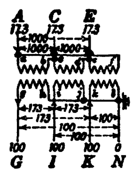
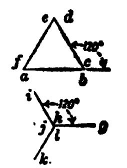
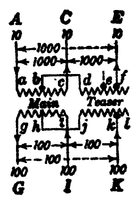
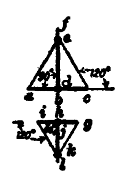
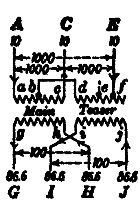
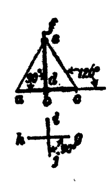
Description L = Capacity of bank. T = Capacity of each transformer.	Diagram Arrows indicate instantaneous currents. Figures indicate r. m. s., volts and amperes.	Vectors Letters correspond to those in the diagrams.
Connection 1. Primary.—Single-phase, 2-wire. Secondary.—Single-phase, 2-wire, double winding connected in multiple. $L = T = 10$ *kv-a.*		
Connection 2. Primary.—Single-phase, 2-wire. Secondary.—Single-phase, 3-wire, neutral grounded, double winding in series. Known as Edison 3-wire system. No current in neutral if load is balanced. Neutral may be omitted for 2-wire circuit. $L = T = 10$ *kv-a.*		
Connection 3. Primary.—Two-phase, 4-wire, non-interconnected. Secondary.—Two-phase, 4-wire, non-interconnected. $L = 2T = 20$ *kv a.*		
Connection 4. Primary.—Two-phase, 4-wire, non-interconnected. Secondary.—Two-phase, 5-wire, interconnected, neutral grounded. No current in neutral if load is balanced. Neutral may be omitted for 4-wire interconnected circuit. $L = 2T = 20$ *kv-a.*		
Connection 5. Primary.—Two-phase, 4-wire, non-interconnected. Secondary.—Two-phase, 5-wire. No current in neutrals if load is balanced. Neutral wires may be omitted for 3-wire circuit. $L = 2T = 20$ *kv-a.*		

TABLE LXIII—*Continued*

STANDARD TRANSFORMER CONNECTIONS

DESCRIPTION L = Capacity of bank. T = Capacity of each transformer.	DIAGRAM Arrows indicate instantaneous currents. Figures indicate r. m. s., volts and amperes.	VECTORS Letters correspond to those in the diagrams.
CONNECTION 6. Primary.—Three-phase, 3-wire, delta. Secondary.—Three-phase, 3-wire, delta. Line current = transformer current $\times \sqrt{3}$. $L = 3T = 30\ kv\text{-}a$.		
CONNECTION 7. Primary.—Three-phase, 3-wire, open delta or "V." Secondary.—Three-phase, 3-wire, open delta or "V." Current in each transformer 30° out of phase with transformer voltage, resulting in reduced capacity. $L = 2T \times .866 = 17.3\ kv\text{-}a$.		
CONNECTION 8. Primary.—Three-phase, 3-wire, delta. Secondary.—Three-phase, 4-wire, star or "Y," neutral grounded. Neutral may be omitted for 3-wire circuit. May be operated with primary star and secondary delta. Star connection for both primary and secondary not recommended unless neutral is grounded. $L = 3T = 30\ kv\text{-}a$.		
CONNECTION 9. Primary.—Three-phase, 3-wire, Scott or "T." Secondary.—Three-phase, 3-wire, Scott or "T." Taps b and h = 50 per cent. Taps c and k = 86.6 per cent. Current in main transformer 30° out of phase with transformer voltage, and voltage in teaser transformer only 86.6 per cent utilized, resulting in reduced capacity. $L = 2T \times .866 = 17.3\ kv\text{-}a$.		
CONNECTION 10. Primary.—Three-phase, 3-wire, Scott or "T." Secondary.—Two-phase, 4-wire, non-interconnected. May be operated with primary 2-phase and secondary 3-phase. Tap b = 50 per cent. Tap e = 86.6 per cent. Three-phase windings operate at reduced capacity as with connection 9 and 2-phase windings designed for 13.4 per cent less capacity to correspond. $L = 2T \times .933 = 17.3\ kv\text{-}a$.		

CHAPTER XXXI

SWITCHING EQUIPMENT, STATION WIRING AND AUXILIARY POWER AND LIGHTING

BY RAYMOND A. HOPKINS

397. Switching Equipment.—Switching equipment, strictly speaking, consists of switches, breakers and other devices used by the station operators for opening or closing electrical circuits and connecting or disconnecting generators, transformers, and other equipment. In many cases, however, switches are fused and circuit-breakers are equipped with trip coils or relays so that they will automatically open their circuits when disturbances occur on the system, and can thus be used both as switching equipment and as protective equipment.

398. Knife Switches.—Knife switches are available in ratings of 0–600 volts and 0–3000 amperes, and in a variety of forms such as 1-, 2-, 3- and 4-pole; single- and double-throw; front- and back-connected; top-, bottom-, or unfused; punched- and milled-clip. They are generally mounted on switchboards or panel boards. They should always be mounted vertically with hinges at the bottom, except that double-throw switches should be mounted horizontally. The blades should be " dead " when open. Switches are used very extensively in the generating station and the substation for controlling excitation, auxiliary power, lighting, and signal circuits.

Fused switches are not recommended above 250 volts d.c. and 500 volts a.c., nor above 1000 amperes at any voltage. Knife switches are rated on the basis of the maximum current they will carry continuously without overheating. For 600-volt circuits, switches are often provided with quick breaks and with barriers. It is often advisable to provide 90° stops to limit the opening of the switches where the panel is crowded.

Clearances which should be allowed in mounting knife switches on switchboards are given in Table LXIV. Dimensions of standard switches are readily obtainable from switch manufacturers.

399. Fuses.—Fuses are of three general types, open-link, plug, and N. E. C. Those for use with knife switches are preferably of the enclosed, N. E. C. type. Standard capacities are as follows:

Capacity of Fuse Clip	Type of Contact	Ampere Capacities of N. E. C. Fuses
1– 30 amp.	Ferrule	1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 12, 15, 20, 25, 30
31– 60 amp.	Ferrule	35, 40, 45, 50, 55, 60
61–100 amp.	Blade	65, 70, 75, 80, 90, 100
101–200 amp.	Blade	110, 120, 130, 140, 150, 160, 170, 180, 190, 200
201–400 amp.	Blade	225, 250, 275, 300, 325, 350, 375, 400
401–600 amp.	Blade	425, 450, 475, 500, 525, 550, 575, 600

400. Enclosed Switches.—Knife switches enclosed in steel boxes arranged to be operated from the outside, commonly known as "Safety Switches," are extensively used throughout the station at motors, on cranes, and at other places apart from the switchboards. They are fused or unfused as desired. The box is generally constructed so that the switch must be opened before the fuses can be reached and so that it can be locked in the open position, both of which features greatly safeguard the maintenance men.

401. Field Switches.—The field switch is designed with a discharge clip which closes while the switch is being opened, so as to discharge the inductive energy from the field through a field-discharge resistance. On account of the flash that usually results from the opening of the field switch, it is often placed on the back of the board and operated by a lever. It is sometimes provided with arcing tips and barriers. The field-transfer switch is a double-throw switch with extra long clips arranged so that the field can be thrown from one bus to another without being opened.

402. Disconnecting Switches.—Disconnecting switches are principally used for isolating oil circuit-breakers, generators, transformers, arresters, and other equipment, as selector switches in conjunction with double buses and single circuit-breakers, and as tie switches between circuits. They are not designed to be opened under load. A disconnecting switch usually consists of a single-pole knife switch arranged with an eye to receive a switch hook by means of which it is opened and closed. It is mounted on porcelain insulators and suitable metal base. A latch to prevent the switch from flying open under short-circuit current is usually necessary except on very small power systems. Gang-operated disconnecting switches are sometimes used to isolate oil circuit-breakers and have the advantage that they can be easily provided with interlocks so that the switches cannot be opened unless the breakers are open.

For outdoor use, disconnecting switches are sometimes provided with horns to direct the arc upward and are then capable of breaking the charging current on short lines or small transformer banks. At 60,000 volts this has successfully been done on lines up to 60 miles long and on transformers up to 7500 kv.-a.

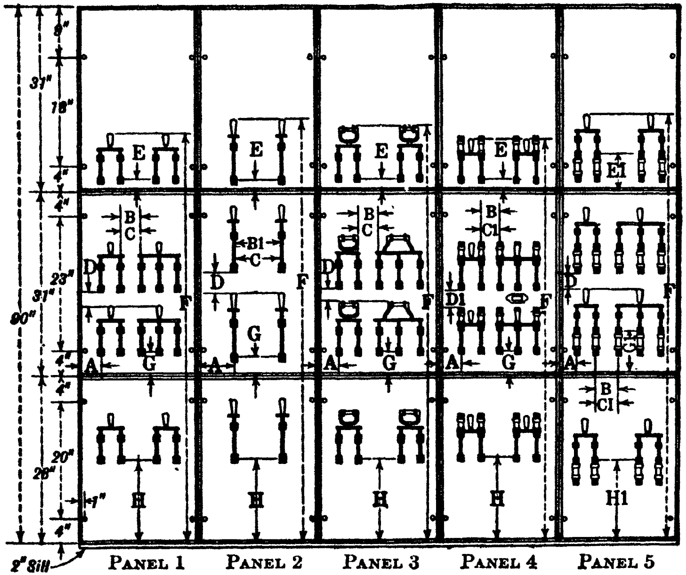
403. Carbon Circuit-breakers.—Carbon circuit-breakers are used most extensively on d.c. circuits such as the excitation and battery circuits in the power station and the d.c. motor generator, rotary converter, and feeder circuits in the substation. They are available in voltages up to 750 and in ampere capacities up to 10,000.

The main contact usually consists of a laminated copper brush bearing against cast copper blocks. The secondary or arcing contacts consist of replaceable carbon or graphite blocks. The secondary contacts close before and open after the main contacts, thus protecting the main contacts from the arc. Breakers of this type are rated on the basis of the maximum current they will carry continuously with a temperature rise on the current-carrying parts, by thermometer, of not more than 30° C. above an ambient temperature of 40° C.

Carbon breakers can be arranged for operation either manually or elec-

TABLE LXIV

LOCATIONS OF SWITCHES ON SWITCHBOARDS



A = Minimum from edge of panel to C.L. of switch blade.

B, B1 = Minimum between C.L. of adjacent switch blades or fuses of opposite polarity and same capacity.

C, C1 = Minimum between C.L. of adjacent switch blades or fuse blocks of same polarity and capacity.

D, D1 = Minimum between top of handle or fuse block and lower edge or hingle clip block, of switch stop when used.

E, E1, G, G1, H, H1 = Minimum from edge of panel to C.L. of clip block.

F = Maximum from bottom edge of panel to top of switch handle.

NOTE.—Line up centers of hinge clips in same horizontal row

PANEL 1—Switches without fuses, 125-volt d.c. or a.c., 250-volt d.c. or a.c., 500-volt a.c.
PANEL 2—Switches without fuses, 600-volt d.c.
PANEL 3—Spade handle switches without fuses, 125-volt d.c. or a.c., 250-volt d.c. or a.c., 500-volt a.c.
PANEL 4—Switches with fuses on contact clips, 250-volt d.c. or a.c., 500-volt a.c.
PANEL 5—Switches with fuses on hinge clips, 250-volt d.c. or a.c., 500-volt a.c.

DIMENSIONS IN INCHES

Amp.	A			B			B1	C	C1		D	D1		E	E1		F	G	G1		** H		** H1	
	A.C. or D.C. 125 V.	D.C. 250 V.	A.C. 500 V. D.C. 600 V.	A.C. or D.C. 125 V.	D.C. 250 V.	A.C. 500 V.	D.C. 600 V.		A.C. or D.C. 250 V. Fuse	A.C. 500 V. Fuse		A.C. or D.C. 250 V. Fuse	A.C. 500 V. Fuse		A.C. or D.C. 250 V. Fuse	A.C. 500 V. Fuse			A.C. or D.C. 250 V. Fuse	A.C. 500 V. Fuse	A.C. 500 V. A.C. or D.C. 250 and 125 V	D.C. 600 V.	A.C. or D.C. 250 V. Fuse	A.C. 500 V. Fuse
30	$2\frac{3}{4}$	3	*	$1\frac{13}{16}$	$2\frac{5}{16}$	*	.....	$1\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{1}{2}$	4	5	$\frac{3}{4}$	3	$6\frac{3}{8}$	75	$\frac{3}{4}$	3	$6\frac{3}{16}$	*$4\frac{3}{4}$	.....	$4\frac{11}{16}$	*$8\frac{1}{2}$
60	$2\frac{3}{4}$	3	†$3\frac{1}{2}$	$2\frac{3}{16}$	$2\frac{15}{16}$	$2\frac{15}{16}$	.....	$1\frac{1}{4}$	$1\frac{1}{4}$	$1\frac{5}{8}$	$1\frac{1}{2}$	4	5	$\frac{13}{16}$	$4\frac{1}{16}$	$6\frac{3}{4}$	75	$\frac{13}{16}$	$4\frac{1}{16}$	$6\frac{9}{16}$	$6\frac{1}{4}$	.....	$5\frac{11}{16}$	$8\frac{7}{8}$
100	$2\frac{13}{16}$	$3\frac{1}{16}$	†$3\frac{9}{16}$	$2\frac{5}{16}$	$3\frac{1}{16}$	$3\frac{1}{16}$	.....	$1\frac{3}{8}$	$1\frac{3}{8}$	$1\frac{5}{8}$	$1\frac{1}{2}$	4	5	$\frac{15}{16}$	7	.....	75	$\frac{15}{16}$	7	9	8	.....	$9\frac{3}{16}$	$11\frac{3}{16}$
200	.....	$3\frac{1}{4}$	$3\frac{3}{4}$	$3\frac{1}{2}$	$3\frac{3}{4}$	$3\frac{3}{4}$	$5\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{7}{8}$	$2\frac{1}{8}$	2	4	5	$1\frac{1}{8}$	.....	.....	75	$1\frac{1}{8}$	$8\frac{1}{2}$	11	10	10	$10\frac{1}{2}$	
300	.....	$3\frac{1}{2}$	4	.....	4	4	$6\frac{1}{8}$	$2\frac{1}{4}$	.....	.....	2	.....	5	$1\frac{1}{4}$	.....	.....	75	$1\frac{1}{4}$	.....	.....	11	11		
400	.....	$3\frac{9}{16}$	$4\frac{1}{16}$	.....	$4\frac{3}{8}$	$4\frac{3}{8}$	$6\frac{3}{8}$	$2\frac{1}{2}$	$2\frac{5}{8}$	$3\frac{1}{8}$	2	4	5	$1\frac{3}{8}$	.....	.....	75	$1\frac{3}{8}$	$10\frac{1}{2}$	$13\frac{1}{2}$	12	12		
600	.....	$3\frac{13}{16}$	$4\frac{5}{16}$	.....	$4\frac{7}{8}$	$4\frac{7}{8}$	$6\frac{7}{8}$	$3\frac{1}{8}$	$3\frac{1}{4}$	$3\frac{3}{4}$	2	4	5	...	.....	.....	72	$1\frac{1}{2}$	$12\frac{5}{8}$	$15\frac{1}{2}$	$13\frac{1}{2}$	$13\frac{1}{2}$		
800	.....	$3\frac{15}{16}$	$4\frac{7}{16}$	.....	$5\frac{3}{8}$	$5\frac{3}{8}$	7	$3\frac{3}{8}$	.....	.....	2	.....	.....	...	.....	.....	72	$1\frac{3}{4}$						
1000	.....	4	$4\frac{1}{2}$	.....	$5\frac{3}{4}$	$5\frac{3}{4}$	$7\frac{1}{4}$	$3\frac{7}{8}$	.....	.....	2	.....	.....	...	.....	.....	72	2						
1200	.....	$4\frac{1}{4}$	$4\frac{3}{4}$	.....	$5\frac{1}{4}$	$5\frac{1}{4}$	$7\frac{1}{4}$	5	.....	.....	2	.....	.....	...	.....	.....	66	$2\frac{1}{4}$						
1600	.....	$4\frac{3}{8}$	$4\frac{7}{8}$	.....	$5\frac{3}{4}$	$5\frac{3}{4}$	8	...	.....	.....	$2\frac{1}{2}$	.....	.....	...	.....	.....	66	$3\frac{1}{2}$						
2000	.....	$4\frac{5}{8}$	$5\frac{1}{8}$	.....	$6\frac{1}{2}$	$6\frac{1}{2}$	$8\frac{1}{2}$	...	.....	.....	$2\frac{1}{2}$	.....	.....	...	.....	.....	66	$3\frac{1}{2}$						
3000	.....	$4\frac{3}{8}$	$4\frac{7}{8}$	.....	$6\frac{1}{2}$	$6\frac{1}{2}$	8	...	.....	.....	$2\frac{1}{2}$	.....	.....	...	.....	.....	66	$3\frac{1}{2}$						

* There is no 30-amp. switch rated 500 volts a.c. without fuse supports.
† There is no 600-volt switch of this capacity.
** Minimum dimensions as given for the lower-capacity switches should be used only when the panel space is limited and this location is necessary.
Data from General Electric Co.

trically. The latter method is often used for excitation breakers when they are located remote from the switchboard. Operating coils can be wound for 125 or 250 volts and should be designed for satisfactory operation at any voltage between 72 per cent and 100 per cent of the normal operating voltage, to take care of variations in the control-battery voltage.

Automatic features include overload, underload, reverse-current and undervoltage trips. Auxiliary switches are usually provided for operating indicating lamps, alarm bells, and interlocks.

Some breakers are arranged with a separate operating handle on each pole and some are arranged to trip free from the operating handle. One or the other of these devices or a series knife switch should always be used so as to prevent the possibility of closing the breaker forcibly against a short circuit.

404. Oil Circuit-breakers.—Oil circuit-breakers are used principally for controlling alternating-current circuits. They may be obtained in ampere capacities from 50 to 4000; for voltages from 220 to 220,000; one-, two-, three- or four-pole; manually or electrically operated; and for indoor or for outdoor service. The selection of the proper breaker for each circuit in a power system involves consideration of several very important factors, such as normal voltage, normal current, abnormal current, interrupting duty, method of operation, arrangement of terminals, space available, accessibility for repairs, altitude of installation above sea level, and temperature.

A breaker is rated in accordance with the Standards of the A.I.E.E. on the basis of (1) the normal r.m.s. voltage of the circuit on which it is intended to operate, (2) the normal r.m.s. current it is designed to carry continuously with a temperature rise of the oil and contacts by thermometer of not over 30° C. above an ambient temperature of 40° C., (3) the normal frequency of the current, and (4) the maximum r.m.s. current at normal voltage which it can interrupt under prescribed conditions at stated intervals a specified number of times. A further rating, which is very important for breakers that are to be operated non-automatically or with a time delay, is (5) the short-time rating, based on the maximum r.m.s. current the breaker will safely carry for a short time, as one, two, three, or five seconds, without excessive heating or mechanical injury.

The prescribed conditions under which the breaker is required to interrupt a current to meet rating (4) above is called the duty cycle of the breaker. The standard duty cycle recommended by the N.E.L.A. and A.I.E.E. is known as the 2–OCO cycle and is as follows: Starting with the breaker open it shall be closed on a short circuit, opened immediately, (i.e., without purposely delayed action) reclosed on the short circuit two minutes after the first closing and opened immediately (i.e., without purposely delayed action). It is understood that the breaker shall interrupt the specified short-circuit current under these conditions without emitting flame, and that after performing its duty cycle inspection shall show that it is in substantially the same mechanical condition and that its main current-carrying parts are in substantially the same condition as at the beginning, although the interrupting ability may be materially reduced. The value of current is taken

during the first half-cycle of arc between contacts during the opening stroke.

The value of the short-circuit current which a breaker will be called upon to interrupt in a particular location in a system depends upon the connected synchronous capacity on the system, the characteristics of the machines, transformers, circuits and other equipment, and the time allowed for the breaker to open after the short-circuit occurs. (See Sec. 355.)

The oil circuit-breaker, as the name implies, is arranged to break the circuit under oil. In the smaller breakers the three poles are arranged in a single oil tank, while in medium and large breakers each pole has a separate tank. Auxiliary contacts are provided to open after and close before the main contacts so as to relieve the main contacts of burning action of the arc. When heavy currents are interrupted by the breaker, the pressure from the heat of the arc is very great and consequently the tank must be made very strong. In some types of breakers, this pressure is utilized to force the contacts to open more rapidly. Provision is often made on heavy-duty breakers to vent the tanks to outdoors to allow safe escape for any gases that may be formed by the arc.

Oil for circuit breakers is obtained from crude petroleum by fractional distillation. Characteristics of commercial circuit-breaker oil are as follows:

Flash point, degrees Centigrade	150 to 190
Burning point, degrees Centigrade	175 to 215
Freezing point, degrees Centigrade	−10 to −30
Weight, pounds per gallon	7.2 to 7.3
Viscosity at 40° C., Saybolt test, seconds	80 to 105
Dielectric strength, between 1 in. disks 0.1 in. apart, volts	20,000 to 30,000

For methods of maintaining the oil dry and clean, reference should be made to the article on transformer oil, Sec. 393.

Accessories for oil circuit-breakers include hand-closing levers for electrically operated breakers, tank-lifting and removing devices, control relay for reducing the duty on the control switch, bushing-type current transformers for relays and ammeters, auxiliary switches for controlling pilot lamps, alarms and interlocks, oil gages, oil-drain and sampling valves, and filter-press connections.

Breakers may be mounted on the switchboard panels or panel pipes, but this is not recommended for voltages higher than 2500 or for ampere ratings higher than 800. Larger breakers are mounted away from the switchboard on pipe or steel framework or in masonry cells. The latter method is used for the main circuits of all but the smallest of power stations and substations, as the masonry cells effectively isolate the various circuits from one another. With the so-called isolated phase arrangement, the three poles of the breaker are placed in separate rooms or on separate floors for more complete isolation. (See " Isolated Phase Arrangement of Switching Equipment " by P. M. Currier and W. T. O'Connell, *General Electric Review*, June, 1923, page 368.) For voltages above 25,000 and occasionally for lower voltages, the breakers

are located outdoors. Generally, more space is available outdoors than indoors and consequently greater clearances can be obtained.

405. Oil Circuit-breaker Specification.—The following brief outline is intended as a guide to the principal items to be covered in an oil circuit breaker specification:

General

Number wanted; reference to latest Standards of the A.I.E.E.; erection by purchaser or contractor; material and workmanship; inspection.

Rating

Voltage; frequency; normal current; maximum current-interrupting capacity; short-time current.

Performance Guarantee

Detailed guarantee of rating. (See Sec. 404.)

Service

Description of service conditions including kind of system; geographical location; altitude; climatic temperature limits; one-line diagram of system showing all interconnections; operation, mounting and arrangement; indoor or outdoor; number and arrangement of poles; location and type of terminals; panel, pipe or cell mounting; cell material, hardware, doors; manually or electrically operated; bell cranks, levers, rods; voltage limits of operating circuit; control relays; control switch; main line operating circuit switch; auxiliary switches to relieve control switch of interrupting duty; arrangement plan and section of station.

Appurtenances

Oil; bushing-type current transformers; auxiliary switches for indicating lamps, alarms, interlocks; capacity and operation of auxiliary switches; tank lifters; emergency operating levers; oil gages; drain and sampling valves; filter connections.

Data to be Submitted by Bidder

Type designation; five-second, three-second, and one-second ampere rating unless specified above; closing and opening currents at normal, maximum and minimum operating-circuit voltages; weight with oil; gallons of oil; kind of oil; outline drawing.

Data to be Submitted by Contractor

Certified detail drawings required for approval, for station design, and for erection.

406. Switchboards.—Switchboards are available in many designs, such as upright, bench or desk, dead-front, truck-type, straight, and curved. The selection depends upon kinds and capacities of apparatus to be controlled, style of switchboard devices to be used, and space available in the station. The main control and instrument board of the power station or substation should be located in a quiet, well-lighted and well-ventilated room for the comfort of the operators who are always in attendance. The exciter circuit-breaker boards, station light and power boards, battery board, and others that do not require constant attendance may be located about the station to secure best economy and convenience. Ample space should be provided for personal safety of employees, both for normal operation and for maintenance and repairs. For all voltages over 150 volts, provision should be made for retiring any portion for working on it without interfering with station operation. Future growth should be provided for by leaving space or blank panels.

The framework for a switchboard consists of a sill, uprights, and braces. A good sill consists of a 6-in. steel channel bolted and grouted to the floor. By making the top of the channel flush with the finished floor, extensions for future panels are not objectionable. A strip of maple laid on top of the channel serves as a good cushion for the panels and also raises the panels 1½ to 2 in. above the floor, giving a clean, attractive appearance. The maple strip is, of course, not needed for boards without sub-panels. It is very important to locate and level the sill perfectly before grouting it in place. The uprights usually consist of 1¼ in. standard black-iron pipes or angle irons. These are braced to the wall or adjacent structure by similar pipes or angles. Screwless bolted fittings assist greatly in obtaining accurate alinement. The space behind the board is usually enclosed by steel grilles with at least two access doors.

Panels are made of slate, marble, ebony asbestos compound, or steel. Slate is the most commonly used material. The standard finish is "Natural black oil finish." Slate does not have particularly good insulating qualities and should never be used for voltages of over 1200 volts. Even for 100 volts it must be carefully selected to avoid metallic veins. Marble may be used for voltages up to 3500 volts and is consequently used extensively for low-voltage disconnecting switch bases. Marble takes a beautiful finish, but the finish is easily spoiled by oil stains and it is very difficult to match existing marble panels. Ebony asbestos is considerably stronger than either slate or marble, and can be safely used with voltages up to 4500 volts. Its texture and color are uniform and it can be finished almost as attractively as can slate. Steel has the advantage of lightness and strength. All current-carrying parts must be insulated from the steel, but this presents very little difficulty on a control board or a truck board.

The Standards of the Electric Power Club give the following standard slate-panel dimensions:

Over-all heights, in inches: 64, 76, 90.
Individual section heights for 76 in. panels, in inches: 48 + 28.

Individual section heights for 90-in. panels, in inches: 62 + 28, 65 + 25, 31 + 31 + 28, 25 + 45 + 20.

Widths, in inches: 16, 20, 24, 28, 32.

Maximum tolerances: height—plus or minus $\frac{1}{16}$ in. per section; width—minus $\frac{1}{16}$ in. per section; thickness—plus or minus $\frac{1}{16}$ in. per section.

Bevel: $\frac{1}{4}$ in. measured on face and edge of panel section.

Equipment for switchboard panels should always be located with respect to its use. Such meters and instruments as the operator must watch during switching operations must be within easy vision. These include bus voltmeters, machine voltmeters, synchroscopes, power-factor indicators, ammeters, and wattmeters. Integrating and curve-drawing meters and relays are usually located near the bottom of the board, or preferably on the rear of the board. Control switches and all devices that require operating must be placed within reach. It is for the purpose of obtaining more space for such equipment that the bench board or control desk is extensively used. This is often extended in a vertical section 10 or 12 in. high at the rear edge to provide still more space within reach of the operator.

407. Switchboard Specification.—The following brief outline is intended as a guide to the principal items to be covered in a switchboard specification. (The reader is further referred to the standard manufacturer's switchboard specification recommended in the Standards of the Electric Power Club.)

General

Reference to Standards of the A.I.E.E.; general description of the system and equipment to be controlled; detailed description of equipment to be controlled by each panel; reference to drawings accompanying the specification which should include complete one-line diagrams and switchboard front views.

Panel Schedule

Complete itemized schedule of all panels, giving meters, instruments, relays, switches, and other devices required on each panel, and breakers, disconnecting switches, instrument transformers, and other equipment required, to be furnished with each panel. This schedule should include every item of material to be provided by the manufacturer. It is suggested that the purpose and requirements of each item be given rather than a too detailed description, as the bidder will then be able to propose his best equipment for such service.

Material not Included

Breakers, instrument transformers, buses, connections, panel wiring, station wiring, erection, and all such items, which are to be supplied by the purchaser. Great care in detailing this paragraph and the preceding paragraph is necessary to define clearly the scope of work.

Panel Construction

General arrangement and dimensions of board; type of sill, framework, and braces; material and finish of panels; extent and shape of enclosing grilles; locations of terminal boards and testing terminals to fit incoming wiring.

Oil Circuit-breakers

If breakers are included they should be fully specified. (See Secs. 404 and 405.)

Instrument Transformers

General type, whether indoor or outdoor, oil-filled or dry; instantaneous ampere capacity of current transformers; schedule or ratios of all transformers to be furnished by purchaser, for manufacturer's use in selecting meter windings; fuses, and resistors for potential transformers.

Mimic Buses

Description of mimic buses; material, as copper, brass, aluminum, or stainless steel for various voltages; disconnecting switches; name plates.

Miscellaneous Instructions

General types of instruments, relays, switches, terminal boards, testing terminals, card holders, unless manufacturer's standards are satisfactory; description of synchronizing requirements; shipment of shunts to other manufacturers to be mounted on other boards; speeds of graphic meters; targets for relays; description of temperature detectors for which instruments must be provided.

Data to be Submitted by the Bidder

Detailed specification giving trade name and designation of each item.

Data to be Supplied by the Contractor

Front view, rear view, and sectional view drawings; wiring diagrams; detail drawings as required for approval, station design, and installation; detailed descriptions and instructions with any items of other than standardized equipment.

408. Station Wiring.—Station wiring consists of all the electrical buses and connections to generators, transformers, switchboards, motor-driven auxiliaries and other electrical equipment. Various kinds of conductors, insulations, supports, and enclosures are used, depending on the particular requirements of each class of wiring.

409. Main Low-voltage Wiring.—The main low-voltage bus into which the generators feed and from which the transformers and feeders are fed is perhaps the most important part of the station wiring, and a ground or short circuit on it would be very serious. The best possible design and construction should, therefore, be used.

The conductor may consist of copper wire, cable, rod, tubing, or bars. For small stations insulated cable or taped copper bar is sometimes used on porcelain insulators, and the bus mounted open above the breakers. Barriers are sometimes added to separate the buses and safeguard employees. For all but very small stations, the low-voltage bus usually consists of copper tubing or bars mounted on porcelain bus supports and enclosed in a masonry structure. Copper bar is usually preferred because (1) the bus can be tapered to suit the loads on various sections, (2) good ventilation can be secured by separating the various bars of each bus by spacers, (3) taps and splices are easily made by bolted joints, and (4) bars can always be added as the station capacity increases. Copper bars of $\frac{1}{8}$, $\frac{1}{4}$, $\frac{3}{8}$, and $\frac{1}{2}$ in. thickness and of various widths from 1 to 10 in. are available. A bar 1 in. wide by $\frac{1}{4}$ in. thick weighs about 1 lb. per foot. (Actually 0.963 lb.) For copper-bar specification see Sec. 415. The Standards of the Electric Power Club specify that the temperature rise shall not exceed 30° C. above an ambient temperature of 40° C., except that for extended bus systems away from apparatus and with unusually heavy contact pressure the rise may be 40° C. above 40° C. The densities in amperes per square inch to give 30° C. rise for $\frac{1}{4}$ in. bars spaced $\frac{1}{4}$ in. apart for 60 cycles per second are given as follows:

Number of Bars	Width of Bars in Inches					
	3	4	5	6	8	10
1	1360	1340	1330	1280	1225	1140
2	1150	1090	1000	950	840	740
3	930	865	750	680	605	555
4	....	750	590	550	490	450
5	....	....	490	460	410	370
6	....	....	420	390	340	300

For alternating currents of over 3000 amperes per phase, consideration should be given to skin effect. (See C. F. Wagner, *Electrical World*, March 18, 1922, page 526.) The joints and connections are very important. The gross contact area should be at least ten times the cross-sectional area of the bar and should be drawfiled or sanded to a net actual contact area of at least 75 per cent of the gross area. Sufficient bolts should be used to assure a contact pressure of not less than 250 lb. per square inch over the entire gross area. Clamps are satisfactory for securing the bus to the bus supports, but for current-carrying joints bolts are likely to distribute the pressure more uniformly, are more positive, and occupy less space.

The selection of suitable bus supports is very important. It is usual to specify a wet flash-over test of two and one-half times working voltage between phases, and a minimum mechanical breaking strength of two or three times the maximum computed short-circuit stress with load applied in any direction through the point of intersection of the center line of the conductor and the axis of the support. (See Sec. 355.) Porcelain bus supports are, of course, stronger under compression than under any other direction of loading,

and in cases where short-circuit stresses are very excessive this advantage is sometimes utilized by surrounding the bus with porcelain supports so that all porcelain is in compression. Other points to be covered in the purchaser's bus-support specification include descriptions of bases, tops, and inserts, and definite instructions as to tests.

Structures for enclosing the main low-voltage buses, disconnecting switches, oil circuit-breakers, instrument transformers, and related equipment may be built of monolithic concrete, pre-cast concrete slabs or blocks, brick, tile, sandstone, soapstone, slate, marble or asbestos lumber, or combinations of these materials. A good construction consists of a solid monolithic concrete main wall with barriers and shelves of pre-cast concrete, brick or tile, keyed into the main wall. Into the main wall can be cast the conduits and bolts or inserts for all bus supports, disconnecting switches, and other equipment. The barriers and shelves are thus not required to carry mechanical loads, but merely serve as isolating and protecting barriers. The structures should be of ample size, not only to allow for the necessary electrical clearances, but to afford convenient space for installing the bus-bars, supports, and joints, and for inspecting and cleaning the porcelains. Doors or grilles should be provided at least 6 ft. high from the floor as a safeguard for employees. In some cases it is considered expedient to cover all conductor compartments with tight fireproof doors.

Reinforcing steel in bus structures, particularly loops around single conductors carrying more than 2000 amperes, should be avoided, as such steel is likely to become heated by induced currents and crack the concrete. Large masses of steel or iron should also be kept at least 12 in. away from such conductors to avoid heating. With voltages of over 10,000 volts, reinforcement in the structures and adjacent floors and walls should be kept at least 12 in. away from the conductors to prevent the spreading of an accidental flash to ground; or, if it cannot be eliminated, it should be welded to the building steel so as to form a definite path to ground and thereby avoid stray currents. One operating company avoids stray ground currents by running a heavy ground wire parallel to each important circuit and connected to the insulator bases and equipment cases. This acts somewhat as a metallic conduit does for a smaller circuit.

For connections from the bus to generators, transformers, and feeders, insulated cables enclosed in fiber conduits are often used. With voltages of over 10,000 volts, however, cables so installed are, under certain conditions, likely to fail on account of localized dielectric loss through the insulation, and for the more important circuits it is often preferable to run the cables open through tunnels, mounted on porcelain insulators between barriers.

410. Main High-voltage Wiring.—With the high-voltage wiring the problem is not one of carrying capacity and mechanical stresses, as with the low voltage wiring, but almost entirely one of insulation. The buses and connections are, therefore, usually made of copper tubing or steel pipe. Such conductors have ample radiating surface and are sufficiently stiff so that but comparatively few supports are needed. Copper tubing is available in standard and extra heavy iron-pipe sizes with a variety of fittings, including splices,

bends, tees, and lugs. The so-called O.D. copper tubing has a thinner wall than the iron-pipe sizes and, therefore, sometimes works out more economically for high-voltage installations where the currents are small and the spans are long. Dimensions and weights of copper tubing are given in Table LXV. For specification see Sec. 415. The current density should be between 1000 and 1200 amperes per square inch, depending upon available ventilation. However, the size is generally dictated by mechanical considerations. For further data on design of outdoor buses see C. A. Mees, *Electrical World,* January 8, 1916, page 86. The conductors can be supported by pin-type or post-type bus supports or by strings of suspension insulators. Barriers are seldom used between phases, but for indoor installations the various buses and circuits are often placed in individual rooms with porcelain bushings where the conductors pass through the walls.

411. Auxiliary Power and Lighting Wiring.—The auxiliary power and lighting wiring includes connections from the house generators and house transformers to the auxiliary power buses, and the distributing system from these buses to the various motor-driven and other electrical equipment. The main cables are usually so large that they must be single-conductor and are, therefore, run in individual fiber conduits. The smaller cables and wiring can be run in iron conduits with the three phases in the same conduit. It is always advisable, if possible, to run the conduits where they will always be dry, and thus avoid the use of lead-covered cable. If in some places leaded cables are necessary they should have potheads if the voltage is 2000 volts or more, and in all cases the lead sheath should be grounded.

Exposed wiring should have a flameproof covering over the insulation and should be substantially mounted on porcelain insulators. Solid wire is often desirable to secure neat appearance.

The general requirements of the National Electrical Code should be followed for all station wiring.

412. Control, Instrument, and Signal Wiring.—The control, instrument, and signal wiring requires perhaps more careful planning than any of the other systems because it is extremely complicated and vitally important. The best insulated wires and cables should be used and these should be carefully installed. Most of this wiring terminates at the main switchboard, and at this point serious congestion is likely to occur. Much trouble can be avoided by the use of a carefully designed control-cable terminal room below the switchboard room.[1] In the terminal room provision can be made for neatly and systematically making all necessary cross connections. In some cases all instrument buses and fuses can be located there, thus greatly simplifying the wiring on the switchboard. Where a conduit room is not feasible, some of its advantages can be gained by providing, behind the board, a large terminal box or trench into which the conduits are brought. Care should always be taken to anticipate and provide for future changes in the wiring.

Multi-conductor cables are sometimes used to great advantage. They may be terminated at one end in suitable boxes in the control-cable terminal

[1] Features of a Switchboard Terminal Room, by R. A. Hopkins, Electrical World, March 13, 1926, p. 561.

TABLE LXV

Weights and Dimensions of Copper Tubing

STANDARD IRON-PIPE SIZES

Nominal Size, Inches	Outside Diameter, Inches	Inside Diameter, Inches	Weight, Pounds Per Foot	Sectional Area, Sq. Inches	Sectional Area, Cir. Mils
½	0.840	0.625	0.95	0.247	315,000
¾	1.050	0.822	1.31	0.335	426,800
1	1.315	1.062	1.79	0.472	601,400
1¼	1.660	1.368	2.63	0.694	884,200
1½	1.900	1.600	3.15	0.825	1,050,000
2	2.375	2.062	4.20	1.091	1,388,800
2½	2.875	2.500	6.04	1.583	2,015,600
3	3.500	3.062	8.72	2.257	2,874,200
3½	4.000	3.500	11.45	2.945	3,750,000
4	4.500	4.000	13.33	3.338	4,250,000

EXTRA HEAVY IRON-PIPE SIZES

Nominal Size, Inches	Outside Diameter, Inches	Inside Diameter, Inches	Weight, Pounds Per Foot	Sectional Area, Sq. Inches	Sectional Area, Cir. Mils
½	0.840	0.542	1.330	0.323	411,800
¾	1.050	0.736	1.750	0.440	560,800
1	1.315	0.951	2.478	0.648	824,800
1¼	1.660	1.272	3.465	0.893	1,137,600
1½	1.900	1.494	4.462	1.082	1,378,000
2	2.375	1.933	5.733	1.496	1,904,100
2½	2.875	2.315	8.715	2.283	2,906,400
3	3.500	2.892	11.760	3.052	3,886,300
3½	4.000	3.358	14.385	3.710	4,723,800
4	4.500	3.818	17.325	4.455	5,672,900

O. D. SIZES

Outside Diameter, Inches	Thickness, Stubs Gage	Inside Diameter, Inches	Weight, Pounds Per Foot	Sectional Area, Sq. Inches	Sectional Area, Cir. Mils
½	18	0.402	0.269	0.071	89,900
¾	17	0.644	0.488	0.116	144,800
1	16	0.870	0.737	0.191	243,100
1¼	15	1.106	1.032	0.266	339,300
1½	14	1.334	1.431	0.369	470,400
2	14	1.834	1.936	0.500	636,400
2½	12	2.282	3.170	0.819	1,042,500
3	11	2.760	4.204	1.086	1,382,400
3½	10	3.232	5.487	1.417	1,804,200
4	10	3.732	6.302	1.627	2,072,200

room and at the other end in similar boxes scattered throughout the station. Such cables are particularly adapted to the signal and telephone systems and can often be used to advantage for control and instrument wiring as well. Extensive grouping of control circuits should be avoided, however, and control and instrument wiring should not be placed in the same cable or conduit.

TABLE LXVI

WORKING TABLE, STANDARD ANNEALED COPPER WIRE

American Wire Gage (B. & S.)

Gage No.	Diameter in Mils	Cross-section		Ohms per 1000 Ft.		Pounds per 1000 Ft.
		Circular Mils	Square Inches	25° C. (=77° F.)	65° C. (=149° F.)	
0000	460.	212,000.	0.166	0.0500	0.0577	641.
000	410.	168,000.	0.132	0.0630	0.0727	508.
00	365.	133,000.	0.105	0.0795	0.0917	403.
0	325.	106,000.	0.0829	0.100	0.116	319.
1	289.	83,700.	0.0657	0.126	0.146	253.
2	258.	66,400.	0.0521	0.159	0.184	201.
3	229.	52,600.	0.0413	0.201	0.232	159.
4	204.	41,700.	0.0328	0.253	0.292	126.
5	182.	33,100.	0.0260	0.319	0.369	100.
6	162.	26,300	0.0206	0.403	0.465	79.5
7	144.	20,800.	0.0164	0.508	0.586	63.0
8	128.	16,500.	0.0130	0.641	0.739	50.0
9	114.	13,100.	0.0103	0.808	0.932	39.6
10	102.	10,400.	0.00815	1.02	1.18	31.4
11	91.	8,230.	0.00647	1.28	1.48	24.9
12	81.	6,530.	0.00513	1.62	1.87	19.8
13	72.	5,180.	0.00407	2.04	2.36	15.7
14	64.	4,110.	0.00323	2.58	2.97	12.4

From Bureau of Standards Circular, No. 31.

413. Wires and Cables.—Conductors for power station use are generally of soft-drawn, annealed copper. Tables LXVI and LXVII give data on solid and stranded bare conductors. Solid conductors are generally used in sizes No. 10 and smaller, and stranded in sizes No. 8 and larger. This, however, is not a fixed rule, as it is often advisable to use solid conductors up to No. 4/0 in exposed work where stiffness is needed, and it is always advisable to use stranded conductors even in small sizes for important control circuits. Flexible stranding is advisable for cables larger than No. 4/0 if pulled into conduits. Rope-core single-conductor cables are sometimes economical where the current exceeds 1000 amperes. They are somewhat larger than standard cables. Sector-type three-conductor cables are occasionally considered in preference to standard cables on account of their slightly smaller diameter. They are somewhat stiffer and require longer bends. Hard drawn or semi-

TABLE LXVII

Bare Concentric-lay Cables of Standard Annealed Copper

Size of Cable		Ohms per 1,000 Ft.		Pounds per 1,000 Ft.	Standard Concentric Stranding			Flexible Concentric Stranding		
Circular Mils	A. W. G. No.	25° C. (= 77° F.)	65° C. (= 149° F.)		Number of Wires	Diameter of Wires, in Mils	Outside Diameter, in Mils	Number of Wires	Diameter of Wires, in Mils	Outside Diameter, in Mils
2,000,000	......	0.00539	0.00622	6,180.	127	125.5	1,631.	169	108.8	1,632.
1,900,000	......	.00568	.00655	5,870.	127	122.3	1,590.	169	106.0	1,590.
1,800,000	......	.00599	.00692	5,560.	127	119.1	1,548.	169	103.2	1,548.
1,700,000	......	.00634	.00732	5,250.	127	115.7	1,504.	169	100.3	1,504.
1,600,000	......	.00674	.00778	4,940.	127	112.2	1,459.	169	97.3	1,460.
1,500,000	......	.00719	.00830	4,630.	91	128.4	1,412.	127	108.7	1,413.
1,400,000	......	.00770	.00889	4,320.	91	124.0	1,364.	127	105.0	1,365.
1,300,000	......	.00830	.00958	4,010.	91	119.5	1,315.	127	101.2	1,315.
1,200,000	......	.00899	.0104	3,710.	91	114.8	1,263.	127	97.2	1,264.
1,100,000	......	.00981	.0114	3,400.	91	109.9	1,209.	127	93.1	1,210.
1,000,000	......	.0108	.0124	3,090.	61	128.0	1,152.	91	104.8	1,153.
950,000	......	.0114	.0131	2,930.	61	124.8	1,123.	91	102.2	1,124.
900,000	......	.0120	.0138	2,780.	61	121.5	1,093.	91	99.4	1,094.
850,000	......	.0127	.0146	2,620.	61	118.0	1,062.	91	96.6	1,063.
800,000	......	.0135	.0156	2,470.	61	114.5	1,031.	91	93.8	1,031.
750,000	......	.0144	.0166	2,320.	61	110.9	998.	91	90.8	999.
700,000	......	.0154	.0178	2,160.	61	107.1	964.	91	87.7	965.
650,000	......	.0166	.0192	2,010.	61	103.2	929.	91	84.5	930.
600,000	......	.0180	.0207	1,850.	61	99.2	893.	91	81.2	893.
550,000	......	.0196	.0226	1,700.	61	95.0	855.	91	77.7	855.
500,000	......	.0216	.0249	1,540.	37	116.2	814.	61	90.5	815.
450,000	......	.0240	.0277	1,390.	37	110.3	772.	61	85.9	773.
400,000	......	.0270	.0311	1,240.	37	104.0	728.	61	81.0	729.
350,000	......	.0308	.0356	1,080.	37	97.3	681.	61	75.7	682.
300,000	......	.0360	.0415	926.	37	90.0	630.	61	70.1	631.
250,000	......	.0431	.0498	772.	37	82.2	575.	61	64.0	576.
212,000	0000	.0509	.0587	653.	19	105.5	528.	37	75.6	533.
168,000	000	.0642	.0741	518.	19	94.0	470.	37	67.3	471.
133,000	00	.0811	.0936	411.	19	83.7	418.	37	60.0	420.
106,000	0	.102	.117	326.	19	74.5	373.	37	53.4	374.
83,700	1	.129	.149	258.	19	66.4	332.	37	47.6	333.
66,400	2	.162	.187	205.	7	97.4	292.	19	59.1	296.
52,600	3	.205	.237	163.	7	86.7	260.	19	52.6	263.
41,700	4	.259	.299	129.	7	77.2	232.	19	46.9	234.
33,100	5	.326	.376	102.	7	68.8	206.	19	41.7	209.
26,300	6	.410	.473	81.0	7	61.2	184.	19	37.2	186.
20,800	7	.519	.599	64.3	7	54.5	164.	19	33.1	166.
16,500	8	.654	.755	51.0	7	48.6	146.	19	29.5	147.

Note.—This table is in accord with standards adopted by the Standards Committee of the American Institute of Electrical Engineers, both in respect to the " Number of wires " and in respect to the correction for increase of resistance and mass due to the twist of the wires. The values given for " Ohms per 1000 feet " and " Pounds per 1000 feet " are 2 per cent greater than for a solid rod of cross section equal to the total cross section of the wires of the cable.

From Bureau of Standards, Circular No. 31.

hard drawn copper is generally used for outdoor work where mechanical strength is important and the added stiffness is not objectionable.

Insulation for electric conductors usually consists of rubber, cambric or paper. Rubber insulation of National Electrical Code quality and thickness (N.E.C. Rubber) is used for practically all station wiring of sizes No. 4/0

TABLE LXVIII

DIMENSIONS: RUBBER-INSULATED WIRES AND CABLES

Table gives approximate diameters, insulation and lead thickness in inches for rubber-insulated double-braided and single-braided lead-covered, single-conductor wires and cables.

Braid thickness used, approximately [illegible] in.

Voltage given is normal working voltage for either N. E. C. rubber or " 30 per cent Para " rubber.

TEST VOLTAGE.—N.E.C. rubber, 1 min.; " 30 per cent Para " rubber, 5 mins. For 600 volts working voltage, see below. All other working voltages 2.5 times normal.

Size	600 Volts					1,500 Volts				2,500 Volts				3,500 Volts				5,000 Volts			
	Braided	Leaded	Insulation	Lead	Test Voltage	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead
16 B. & S.	0.16	0.27	[illegible]	[illegible]	1500	0.36	0.43	[illegible]	[illegible]	0.44	0.51	[illegible]	[illegible]	0.50	0.57	[illegible]	[illegible]				
14 "	0.22	0.31	[illegible]	[illegible]	1500	0.37	0.44	[illegible]	[illegible]	0.45	0.52	[illegible]	[illegible]	0.51	0.58	[illegible]	[illegible]				
12 "	0.25	0.33	[illegible]	[illegible]	1500	0.39	C.46	[illegible]	[illegible]	0.47	0.54	[illegible]	[illegible]	0.53	0.60	[illegible]	[illegible]				
10 "	0.27	0.38	[illegible]	[illegible]	1500	0.43	0.50	[illegible]	[illegible]	0.50	0.57	[illegible]	[illegible]	0.56	0.63	[illegible]	[illegible]				
8 "	0.30	0.41	[illegible]	[illegible]	1500	0.46	0.53	[illegible]	[illegible]	0.53	0.60	[illegible]	[illegible]	0.59	0.66	[illegible]	[illegible]	0.68	0.74	[illegible]	[illegible]
6 "	0.38	0.47	[illegible]	[illegible]	2000	0.52	0.58	[illegible]	[illegible]	0.62	0.69	[illegible]	[illegible]	0.65	0.72	[illegible]	[illegible]	0.72	0.81	[illegible]	[illegible]
4 "	0.42	0.52	[illegible]	[illegible]	2000	0.58	0.64	[illegible]	[illegible]	0.67	0.74	[illegible]	[illegible]	0.70	0.77	[illegible]	[illegible]	0.77	0.86	[illegible]	[illegible]
3 "	0.45	0.55	[illegible]	[illegible]	2000	0.62	0.68	[illegible]	[illegible]	0.70	0.77	[illegible]	[illegible]	0.73	0.80	[illegible]	[illegible]	0.79	0.88	[illegible]	[illegible]
2 "	0.49	0.58	[illegible]	[illegible]	2000	0.67	0.73	[illegible]	[illegible]	0.73	0.83	[illegible]	[illegible]	0.76	0.86	[illegible]	[illegible]	0.83	0.92	[illegible]	[illegible]
1 "	0.55	0.65	[illegible]	[illegible]	2500	0.75	0.85	[illegible]	[illegible]	0.81	0.91	[illegible]	[illegible]	0.81	0.91	[illegible]	[illegible]	0.87	0.96	[illegible]	[illegible]
0 "	0.63	0.69	[illegible]	[illegible]	2500	0.78	0.88	[illegible]	[illegible]	0.84	0.94	[illegible]	[illegible]	0.84	0.94	[illegible]	[illegible]	0.92	1.01	[illegible]	[illegible]
00 "	0.66	0.74	[illegible]	[illegible]	2500	0.83	0.93	[illegible]	[illegible]	0.89	0.99	[illegible]	[illegible]	0.89	0.99	[illegible]	[illegible]	0.97	1.06	[illegible]	[illegible]
000 "	0.72	0.78	[illegible]	[illegible]	2500	0.88	0.98	[illegible]	[illegible]	0.94	1.04	[illegible]	[illegible]	0.94	1.04	[illegible]	[illegible]	1.02	1.15	[illegible]	[illegible]
0000 "	0.78	0.85	[illegible]	[illegible]	2500	0.95	1.08	[illegible]	[illegible]	1.01	1.14	[illegible]	[illegible]	1.01	1.14	[illegible]	[illegible]	1.08	1.21	[illegible]	[illegible]
250,000 C. M.	0.91	0.98	[illegible]	[illegible]	3000	1.05	1.18	[illegible]	[illegible]	1.08	1.21	[illegible]	[illegible]	1.09	1.22	[illegible]	[illegible]	1.12	1.25	[illegible]	[illegible]
300,000 "	0.94	1.02	[illegible]	[illegible]	3000	1.09	1.22	[illegible]	[illegible]	1.12	1.25	[illegible]	[illegible]	1.15	1.28	[illegible]	[illegible]	1.18	1.31	[illegible]	[illegible]
350,000 "	1.00	1.09	[illegible]	[illegible]	3000	1.13	1.26	[illegible]	[illegible]	1.16	1.29	[illegible]	[illegible]	1.20	1.33	[illegible]	[illegible]	1.23	1.36	[illegible]	[illegible]
400,000 "	1.06	1.11	[illegible]	[illegible]	3000	1.18	1.31	[illegible]	[illegible]	1.21	1.34	[illegible]	[illegible]	1.25	1.38	[illegible]	[illegible]	1.28	1.41	[illegible]	[illegible]
450,000 "	1.10	1.16	[illegible]	[illegible]	3000	1.23	1.36	[illegible]	[illegible]	1.26	1.39	[illegible]	[illegible]	1.29	1.42	[illegible]	[illegible]	1.32	1.45	[illegible]	[illegible]
500,000 "	1.16	1.23	[illegible]	[illegible]	3000	1.28	1.41	[illegible]	[illegible]	1.31	1.44	[illegible]	[illegible]	1.34	1.47	[illegible]	[illegible]	1.37	1.50	[illegible]	[illegible]
550,000 "	1.25	1.35	[illegible]	[illegible]	3500	1.35	1.48	[illegible]	[illegible]	1.35	1.48	[illegible]	[illegible]	1.42	1.55	[illegible]	[illegible]	1.42	1.55	[illegible]	[illegible]
600,000 "	1.28	1.38	[illegible]	[illegible]	3500	1.37	1.50	[illegible]	[illegible]	1.37	1.50	[illegible]	[illegible]	1.45	1.58	[illegible]	[illegible]	1.45	1.58	[illegible]	[illegible]
650,000 "	1.31	1.41	[illegible]	[illegible]	3500	1.41	1.54	[illegible]	[illegible]	1.41	1.54	[illegible]	[illegible]	1.48	1.60	[illegible]	[illegible]	1.48	1.60	[illegible]	[illegible]
700,000 "	1.35	1.44	[illegible]	[illegible]	3500	1.45	1.58	[illegible]	[illegible]	1.45	1.58	[illegible]	[illegible]	1.51	1.64	[illegible]	[illegible]	1.51	1.64	[illegible]	[illegible]
750,000 "	1.38	1.47	[illegible]	[illegible]	3500	1.49	1.62	[illegible]	[illegible]	1.49	1.62	[illegible]	[illegible]	1.55	1.71	[illegible]	[illegible]	1.55	1.71	[illegible]	[illegible]
800,000 "	1.41	1.50	[illegible]	[illegible]	3500	1.52	1.68	[illegible]	[illegible]	1.52	1.68	[illegible]	[illegible]	1.59	1.75	[illegible]	[illegible]	1.59	1.75	[illegible]	[illegible]
850,000 "	1.47	1.55	[illegible]	[illegible]	3500	1.54	1.70	[illegible]	[illegible]	1.54	1.70	[illegible]	[illegible]	1.61	1.77	[illegible]	[illegible]	1.61	1.77	[illegible]	[illegible]
900,000 "	1.50	1.56	[illegible]	[illegible]	3500	1.58	1.74	[illegible]	[illegible]	1.58	1.74	[illegible]	[illegible]	1.64	1.80	[illegible]	[illegible]	1.64	1.80	[illegible]	[illegible]
950,000 "	1.53	1.60	[illegible]	[illegible]	3500	1.60	1.76	[illegible]	[illegible]	1.60	1.76	[illegible]	[illegible]	1.67	1.83	[illegible]	[illegible]	1.67	1.83	[illegible]	[illegible]
1,000,000 "	1.56	1.64	[illegible]	[illegible]	3500	1.65	1.81	[illegible]	[illegible]	1.65	1.81	[illegible]	[illegible]	1.70	1.87	[illegible]	[illegible]	1.70	1.87	[illegible]	[illegible]
1,250,000 "	1.72	1.81	[illegible]	[illegible]	3500	1.80	1.96	[illegible]	[illegible]	1.80	1.96	[illegible]	[illegible]	1.86	2.03	[illegible]	[illegible]	1.93	2.09	[illegible]	[illegible]
1,500,000 "	1.87	1.97	[illegible]	[illegible]	3500	1.93	2.10	[illegible]	[illegible]	1.93	2.10	[illegible]	[illegible]	2.00	2.16	[illegible]	[illegible]	2.06	2.22	[illegible]	[illegible]
1,750,000 "	2.00	2.16	[illegible]	[illegible]	3500	2.04	2.21	[illegible]	[illegible]	2.04	2.21	[illegible]	[illegible]	2.11	2.27	[illegible]	[illegible]	2.17	2.33	[illegible]	[illegible]
2,000,000 "	2.09	2.25	[illegible]	[illegible]	3500	2.15	2.32	[illegible]	[illegible]	2.15	2.32	[illegible]	[illegible]	2.22	2.38	[illegible]	[illegible]	2.28	2.44	[illegible]	[illegible]

TABLE LXVIII—*Continued*

Size	7,000 Volts				9,000 Volts				11,000 Volts				13,000 Volts				15,000 Volts				17,000 Volts			
	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead
8 B. & S.	0.83	0.93	$\frac{1}{4}$	$\frac{5}{64}$	0.95	1.04	$\frac{5}{16}$	$\frac{5}{64}$	1.02	1.11	$\frac{11}{32}$	$\frac{5}{64}$												
6 "	0.86	0.95	$\frac{1}{4}$	$\frac{5}{64}$	0.99	1.08	$\frac{5}{16}$	$\frac{5}{64}$	1.05	1.14	$\frac{11}{32}$	$\frac{5}{64}$												
4 "	0.90	0.99	$\frac{1}{4}$	$\frac{5}{64}$	1.03	1.12	$\frac{5}{16}$	$\frac{5}{64}$	1.08	1.21	$\frac{11}{32}$	$\frac{3}{32}$	1.15	1.28	$\frac{12}{32}$	$\frac{3}{32}$	1.22	1.35	$\frac{13}{32}$	$\frac{3}{32}$	1.28	1.41	$\frac{7}{16}$	$\frac{3}{32}$
3 "	0.92	1.01	$\frac{1}{4}$	$\frac{5}{64}$	1.05	1.14	$\frac{5}{16}$	$\frac{5}{64}$	1.11	1.24	$\frac{11}{32}$	$\frac{3}{32}$	1.17	1.30	$\frac{12}{32}$	$\frac{3}{32}$	1.24	1.37	$\frac{13}{32}$	$\frac{3}{32}$	1.30	1.43	$\frac{7}{16}$	$\frac{3}{32}$
2 "	0.96	1.05	$\frac{1}{4}$	$\frac{5}{64}$	1.09	1.22	$\frac{5}{16}$	$\frac{3}{32}$	1.15	1.28	$\frac{11}{32}$	$\frac{3}{32}$	1.21	1.34	$\frac{12}{32}$	$\frac{3}{32}$	1.28	1.41	$\frac{13}{32}$	$\frac{3}{32}$	1.34	1.47	$\frac{7}{16}$	$\frac{3}{32}$
1 "	1.00	1.09	$\frac{1}{4}$	$\frac{5}{64}$	1.13	1.26	$\frac{5}{16}$	$\frac{3}{32}$	1.19	1.32	$\frac{11}{32}$	$\frac{3}{32}$	1.25	1.38	$\frac{12}{32}$	$\frac{3}{32}$	1.32	1.45	$\frac{13}{32}$	$\frac{3}{32}$	1.38	1.51	$\frac{7}{16}$	$\frac{3}{32}$
0 "	1.05	1.14	$\frac{1}{4}$	$\frac{5}{64}$	1.18	1.30	$\frac{5}{16}$	$\frac{3}{32}$	1.24	1.37	$\frac{11}{32}$	$\frac{3}{32}$	1.30	1.43	$\frac{12}{32}$	$\frac{3}{32}$	1.37	1.50	$\frac{13}{32}$	$\frac{3}{32}$	1.43	1.56	$\frac{7}{16}$	$\frac{3}{32}$
00 "	1.10	1.23	$\frac{1}{4}$	$\frac{3}{32}$	1.23	1.36	$\frac{5}{16}$	$\frac{3}{32}$	1.29	1.42	$\frac{11}{32}$	$\frac{3}{32}$	1.35	1.48	$\frac{12}{32}$	$\frac{3}{32}$	1.42	1.55	$\frac{13}{32}$	$\frac{3}{32}$	1.48	1.61	$\frac{7}{16}$	$\frac{3}{32}$
000 "	1.15	1.28	$\frac{1}{4}$	$\frac{3}{32}$	1.28	1.41	$\frac{5}{16}$	$\frac{3}{32}$	1.34	1.47	$\frac{11}{32}$	$\frac{3}{32}$	1.40	1.53	$\frac{12}{32}$	$\frac{3}{32}$	1.47	1.60	$\frac{13}{32}$	$\frac{3}{32}$	1.53	1.66	$\frac{7}{16}$	$\frac{3}{32}$
0000 "	1.19	1.32	$\frac{1}{4}$	$\frac{3}{32}$	1.32	1.45	$\frac{5}{16}$	$\frac{3}{32}$	1.39	1.52	$\frac{11}{32}$	$\frac{3}{32}$	1.45	1.58	$\frac{12}{32}$	$\frac{3}{32}$	1.51	1.64	$\frac{13}{32}$	$\frac{3}{32}$	1.57	1.73	$\frac{7}{16}$	$\frac{7}{64}$
250,000 C. M.	1.25	1.38	$\frac{1}{4}$	$\frac{3}{32}$	1.38	1.51	$\frac{5}{16}$	$\frac{3}{32}$	1.45	1.58	$\frac{11}{32}$	$\frac{3}{32}$	1.51	1.64	$\frac{12}{32}$	$\frac{3}{32}$	1.57	1.70	$\frac{13}{32}$	$\frac{7}{64}$	1.63	1.79	$\frac{7}{16}$	$\frac{7}{64}$
300,000 "	1.31	1.44	$\frac{1}{4}$	$\frac{3}{32}$	1.44	1.57	$\frac{5}{16}$	$\frac{3}{32}$	1.51	1.64	$\frac{11}{32}$	$\frac{3}{32}$	1.57	1.73	$\frac{12}{32}$	$\frac{7}{64}$	1.63	1.79	$\frac{13}{32}$	$\frac{7}{64}$	1.69	1.85	$\frac{7}{16}$	$\frac{7}{64}$
350,000 "	1.36	1.49	$\frac{1}{4}$	$\frac{3}{32}$	1.49	1.62	$\frac{5}{16}$	$\frac{3}{32}$	1.56	1.72	$\frac{11}{32}$	$\frac{7}{64}$	1.62	1.78	$\frac{12}{32}$	$\frac{7}{64}$	1.68	1.84	$\frac{13}{32}$	$\frac{7}{64}$	1.74	1.90	$\frac{7}{16}$	$\frac{7}{64}$
400,000 "	1.39	1.52	$\frac{1}{4}$	$\frac{3}{32}$	1.52	1.68	$\frac{5}{16}$	$\frac{7}{64}$	1.59	1.75	$\frac{11}{32}$	$\frac{7}{64}$	1.65	1.81	$\frac{12}{32}$	$\frac{7}{64}$	1.71	1.87	$\frac{13}{32}$	$\frac{7}{64}$	1.77	1.96	$\frac{7}{16}$	$\frac{1}{8}$
450,000 "	1.45	1.58	$\frac{1}{4}$	$\frac{3}{32}$	1.58	1.74	$\frac{5}{16}$	$\frac{7}{64}$	1.65	1.81	$\frac{11}{32}$	$\frac{7}{64}$	1.71	1.87	$\frac{12}{32}$	$\frac{7}{64}$	1.77	1.93	$\frac{13}{32}$	$\frac{1}{8}$	1.83	2.02	$\frac{7}{16}$	$\frac{1}{8}$
500,000 "	1.49	1.62	$\frac{1}{4}$	$\frac{3}{32}$	1.62	1.78	$\frac{5}{16}$	$\frac{7}{64}$	1.69	1.85	$\frac{11}{32}$	$\frac{7}{64}$	1.75	1.91	$\frac{12}{32}$	$\frac{7}{64}$	1.81	2.00	$\frac{13}{32}$	$\frac{1}{8}$	1.87	2.06	$\frac{7}{16}$	$\frac{1}{8}$
550,000 "	1.54	1.70	$\frac{1}{4}$	$\frac{7}{64}$	1.67	1.83	$\frac{5}{16}$	$\frac{7}{64}$	1.74	1.90	$\frac{11}{32}$	$\frac{7}{64}$	1.80	1.98	$\frac{12}{32}$	$\frac{1}{8}$	1.86	2.05	$\frac{13}{32}$	$\frac{1}{8}$	1.92	2.11	$\frac{7}{16}$	$\frac{1}{8}$
600,000 "	1.57	1.73	$\frac{1}{4}$	$\frac{7}{64}$	1.70	1.86	$\frac{5}{16}$	$\frac{7}{64}$	1.77	1.93	$\frac{11}{32}$	$\frac{1}{8}$	1.83	2.02	$\frac{12}{32}$	$\frac{1}{8}$	1.89	2.08	$\frac{13}{32}$	$\frac{1}{8}$	1.95	2.14	$\frac{7}{16}$	$\frac{1}{8}$
650,000 "	1.61	1.77	$\frac{1}{4}$	$\frac{7}{64}$	1.74	1.90	$\frac{5}{16}$	$\frac{7}{64}$	1.81	2.00	$\frac{11}{32}$	$\frac{1}{8}$	1.87	2.06	$\frac{12}{32}$	$\frac{1}{8}$	1.93	2.12	$\frac{13}{32}$	$\frac{1}{8}$	1.99	2.18	$\frac{7}{16}$	$\frac{1}{8}$
700,000 "	1.64	1.80	$\frac{1}{4}$	$\frac{7}{64}$	1.77	1.96	$\frac{5}{16}$	$\frac{1}{8}$	1.84	2.03	$\frac{11}{32}$	$\frac{1}{8}$	1.90	2.09	$\frac{12}{32}$	$\frac{1}{8}$	1.96	2.15	$\frac{13}{32}$	$\frac{1}{8}$	2.02	2.21	$\frac{7}{16}$	$\frac{1}{8}$
750,000 "	1.67	1.83	$\frac{1}{4}$	$\frac{7}{64}$	1.81	2.00	$\frac{5}{16}$	$\frac{1}{8}$	1.87	2.06	$\frac{11}{32}$	$\frac{1}{8}$	1.93	2.12	$\frac{12}{32}$	$\frac{1}{8}$	1.99	2.18	$\frac{13}{32}$	$\frac{1}{8}$	2.05	2.24	$\frac{7}{16}$	$\frac{1}{8}$
800,000 "	1.72	1.88	$\frac{1}{4}$	$\frac{7}{64}$	1.84	2.03	$\frac{5}{16}$	$\frac{1}{8}$	1.91	2.10	$\frac{11}{32}$	$\frac{1}{8}$	1.97	2.16	$\frac{12}{32}$	$\frac{1}{8}$	2.03	2.22	$\frac{13}{32}$	$\frac{1}{8}$	2.09	2.28	$\frac{7}{16}$	$\frac{1}{8}$
850,000 "	1.74	1.90	$\frac{1}{4}$	$\frac{7}{64}$	1.87	2.06	$\frac{5}{16}$	$\frac{1}{8}$	1.94	2.13	$\frac{11}{32}$	$\frac{1}{8}$	2.00	2.19	$\frac{12}{32}$	$\frac{1}{8}$	2.06	2.25	$\frac{13}{32}$	$\frac{1}{8}$	2.12	2.31	$\frac{7}{16}$	$\frac{1}{8}$
900,000 "	1.77	1.96	$\frac{1}{4}$	$\frac{1}{8}$	1.90	2.09	$\frac{5}{16}$	$\frac{1}{8}$	1.97	2.16	$\frac{11}{32}$	$\frac{1}{8}$	2.03	2.22	$\frac{12}{32}$	$\frac{1}{8}$	2.09	2.28	$\frac{13}{32}$	$\frac{1}{8}$	2.15	2.34	$\frac{7}{16}$	$\frac{1}{8}$
950,000 "	1.80	1.99	$\frac{1}{4}$	$\frac{1}{8}$	1.93	2.12	$\frac{5}{16}$	$\frac{1}{8}$	2.00	2.19	$\frac{11}{32}$	$\frac{1}{8}$	2.06	2.25	$\frac{12}{32}$	$\frac{1}{8}$	2.12	2.31	$\frac{13}{32}$	$\frac{1}{8}$	2.18	2.37	$\frac{7}{16}$	$\frac{1}{8}$
1,000,000 "	1.83	2.02	$\frac{1}{4}$	$\frac{1}{8}$	1.96	2.15	$\frac{5}{16}$	$\frac{1}{8}$	2.03	2.22	$\frac{11}{32}$	$\frac{1}{8}$	2.09	2.28	$\frac{12}{32}$	$\frac{1}{8}$	2.15	2.34	$\frac{13}{32}$	$\frac{1}{8}$	2.21	2.40	$\frac{7}{16}$	$\frac{1}{8}$
1,250,000 "	1.96	2.12	$\frac{9}{32}$	$\frac{1}{8}$	2.13	2.29	$\frac{5}{16}$	$\frac{1}{8}$	2.19	2.35	$\frac{11}{32}$	$\frac{1}{8}$	2.26	2.42	$\frac{12}{32}$	$\frac{1}{8}$	2.32	2.48	$\frac{13}{32}$	$\frac{1}{8}$	2.38	2.50	$\frac{7}{16}$	$\frac{1}{8}$
1,500,000 "	2.18	2.34	$\frac{9}{32}$	$\frac{1}{8}$	2.26	2.42	$\frac{5}{16}$	$\frac{1}{8}$	2.32	2.48	$\frac{11}{32}$	$\frac{1}{8}$	2.39	2.55	$\frac{12}{32}$	$\frac{1}{8}$	2.45	2.61	$\frac{13}{32}$	$\frac{1}{8}$	2.51	2.67	$\frac{7}{16}$	$\frac{1}{8}$
1,750,000 "	2.31	2.47	$\frac{9}{32}$	$\frac{1}{8}$	2.36	2.52	$\frac{5}{16}$	$\frac{1}{8}$	2.42	2.58	$\frac{11}{32}$	$\frac{1}{8}$	2.49	2.65	$\frac{12}{32}$	$\frac{1}{8}$	2.55	2.75	$\frac{13}{32}$	$\frac{1}{8}$	2.58	2.74	$\frac{7}{16}$	$\frac{1}{8}$
2,000,000 "	2.41	2.57	$\frac{9}{32}$	$\frac{1}{8}$	2.48	2.64	$\frac{5}{16}$	$\frac{1}{8}$	2.54	2.70	$\frac{11}{32}$	$\frac{1}{8}$	2.61	2.77	$\frac{12}{32}$	$\frac{1}{8}$	2.66	2.82	$\frac{13}{32}$	$\frac{1}{8}$	2.70	2.86	$\frac{7}{16}$	$\frac{1}{8}$

TABLE LXIX

Weights: Rubber-insulated Wires and Cables

Table gives approximate weights in pounds per thousand feet for wires and cables.

Unit Weights Used

Standard stranded annealed copper........................566 lb. per cu. ft.
Rubber (either N.E.C. or 30 per cent Para)..............110 lb. per cu. ft.
Varnished cambric (as laid on the cable)................. 71 lb. per cu. ft.
Weatherproof braid (as laid on the cable)................ 70 lb. per cu. ft.
Flameproof braid (as laid on the cable)..................210 lb. per cu. ft.
Lead—pure...710 lb. per cu. ft.

Notes

1. Weights based on standard stranded annealed copper.
2. Weights based on double braid for braided conductors.
3. Weights based on single braid or tape between insulation and lead for leaded conductors.
4. Weights will apply without appreciable error to solid conductor wires and cables.

Size	600 Volts			1,500 Volts			2,500 Volts			3,500 Volts		
	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead
16 B. & S.	18.8	27.2	181	68	120	407	96	164	521	124	200	590
14 "	33.5	51.1	226	80	138	457	111	187	571	139	223	640
12 "	45.9	68.9	249	93	153	509	128	212	623	160	254	692
10 "	62.8	84.8	364	115	185	572	150	242	686	184	284	755
8 "	84.6	110	430	143	219	642	179	279	750	213	321	831
6 "	135.6	169	523	205	297	729	254	384	912	276	405	930
4 "	190	228	624	273	331	834	328	474	1,017	346	492	1,047
3 "	234	280	703	330	460	958	382	526	1,114	403	571	1,132
2 "	284	345	741	386	532	1,063	430	592	1,365	461	637	1,378
1 "	363	416	906	476	636	1,377	526	702	1,542	530	714	1,554
0 "	450	542	1,019	564	740	1,535	626	818	1,700	620	812	1,706
00 "	539	616	1,167	665	849	1,716	727	927	1,854	727	927	1,854
000 "	664	764	1,307	792	992	1,961	853	1,067	2,213	859	1,073	2,069
0000 "	816	938	1,550	958	1,196	2,210	1,030	1,268	2,540	1,024	1,262	2,552
250,000 C. M.	1,016	1,216	2,080	1,165	1,471	2,722	1,182	1,450	2,881	1,202	1,462	2,896
300,000 "	1,170	1,346	2,262	1,348	1,676	3,005	1,356	1,640	3,125	1,398	1,682	3,164
350,000 "	1,348	1,548	2,502	1,532	1,884	3,288	1,543	1,833	3,320	1,587	1,893	3,462
400,000 "	1,538	1,774	2,696	1,722	2,098	3,577	1,719	2,017	3,580	1,767	2,089	3,724
450,000 "	1,694	1,930	2,896	1,891	2,281	3,835	1,893	2,215	3,850	1,943	2,281	3,970
500,000 "	1,876	2,152	3,206	2,061	2,467	4,105	2,062	2,398	4,108	2,116	2,476	4,234
550,000 "	2,116	2,444	3,884	2,241	2,663	4,439	2,238	2,582	4,360	2,341	2,729	4,544
600,000 "	2,267	2,573	4,150	2,479	2,969	4,916	2,360	2,762	4,565	2,519	2,902	4,802
650,000 "	2,444	2,760	4,282	2,676	3,188	5,205	2,592	2,962	4,776	2,701	3,099	5,019
700,000 "	2,608	2,940	4,466	2,838	3,366	5,451	2,761	3,135	4,983	2,879	3,285	5,259
750,000 "	2,778	3,118	4,678	3,028	3,589	5,764	2,941	3,331	5,257	3,061	3,481	5,839
800,000 "	2,932	3,256	4,868	3,197	3,763	6,004	3,107	3,505	5,488	3,239	3,661	6,136
850,000 "	3,129	3,535	5,104	3,367	3,949	6,244	3,279	3,685	6,049	3,414	3,850	6,343
900,000 "	3,305	3,719	5,279	3,546	4,142	6,299	3,455	3,869	6,299	3,590	4,034	6,554
950,000 "	3,465	3,887	5,531	3,726	4,346	6,500	3,627	4,049	6,500	3,756	4,208	6,755
1,000,000 "	3,635	4,065	5,761	3,905	4,539	6,774	3,814	4,266	6,735	3,932	4,392	6,984
1,250,000 "	4,509	4,945	7,592	4,754	5,450	8,186	4,657	5,171	8,135	4,792	5,336	8,902
1,500,000 "	5,378	5,950	8,832	5,583	6,325	9,124	5,493	6,061	10,637	5,493	6,223	9,904
1,750,000 "	6,238	6,910	10,214	6,462	7,282	10,150	6,317	6,907	11,608	6,317	7,126	11,933
2,000,000 "	7,032	7,670	11,130	7,282	8,142	11,244	7,169	7,827	12,745	7,169	8,046	12,060

TABLE LXIX—*Continued*

Size	5,000 Volts			7,000 Volts			9,000 Volts			11,000 Volts		
	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead
8 B. & S.	277	429	1,062	392	606	1,452	514	744	1,716	565	825	1,788
6 "	330	492	1,251	456	678	1,533	572	810	1,815	653	921	1,977
4 "	399	567	1,368	526	756	1,644	657	909	1,944	747	1,023	2,373
3 "	448	622	1,453	588	826	1,729	725	985	2,029	809	1,093	2,503
2 "	519	703	1,552	658	904	1,846	789	1,057	2,476	873	1,165	2,659
1 "	588	780	1,674	738	990	1,956	870	1,146	2,637	971	1,269	2,793
0 "	689	911	1,850	832	1,100	2,120	991	1,289	2,780	1,081	1,403	2,993
00 "	800	1,038	2,004	945	1,221	2,673	1,116	1,446	3,036	1,207	1,557	3,249
000 "	936	1,196	2,192	1,092	1,376	2,894	1,272	1,616	3,269	1,358	1,718	3,476
0000 "	1,105	1,373	2,747	1,261	1,553	3,104	1,453	1,805	3,512	1,567	1,943	3,731
250,000 C. M.	1,258	1,534	3,007	1,431	1,753	3,343	1,624	1,984	3,769	1,727	2,125	4,006
300,000 "	1,441	1,739	3,269	1,617	1,955	3,695	1,827	2,225	4,100	1,941	2,363	4,286
350,000 "	1,621	1,935	3,543	1,816	2,160	3,957	2,031	2,445	4,368	2,150	2,568	4,929
400,000 "	1,801	2,131	3,805	1,992	2,344	4,180	2,213	2,635	4,942	2,354	2,806	5,236
450,000 "	1,986	2,332	4,051	2,190	2,566	4,462	2,415	2,845	5,251	2,548	3,016	5,473
500,000 "	2,166	2,518	4,321	2,372	2,764	4,720	2,616	3,052	5,533	2,731	3,205	5,743
550,000 "	2,357	2,747	4,595	2,563	2,969	5,257	2,806	3,266	5,825	2,929	3,419	6,103
600,000 "	2,529	2,927	4,802	2,751	3,173	5,513	2,996	3,464	6,071	3,119	3,617	6,556
650,000 "	2,705	3,111	5,013	2,942	3,378	5,781	3,171	3,645	6,335	3,323	3,837	6,912
700,000 "	2,883	3,297	5,243	3,114	3,558	6,039	3,374	3,858	6,960	3,561	4,023	7,254
750,000 "	3,062	3,490	5,827	3,296	3,748	6,295	3,549	4,039	7,228	3,698	4,234	7,510
800,000 "	3,234	3,670	6,079	3,472	3,940	6,559	3,725	4,231	7,474	3,880	4,432	7,780
850,000 "	3,406	3,850	6,286	3,659	4,141	6,815	3,903	4,417	7,720	4,070	4,630	8,044
900,000 "	3,582	4,034	6,503	3,847	4,337	7,373	4,103	4,625	7,994	4,258	4,826	8,279
950,000 "	3,760	4,220	6,704	4,023	4,529	7,658	4,276	4,804	8,199	4,423	4,997	8,486
1,000,000 "	3,936	4,404	7,017	4,205	4,719	7,926	4,490	5,034	8,432	4,617	5,199	8,721
1,250,000 "	4,924	5,492	8,714	5,267	5,957	8,978	5,407	6,089	9,494	5,544	6,248	9,638
1,500,000 "	5,804	6,424	9,808	6,112	6,832	10,108	6,276	7,012	10,747	6,455	7,213	11,047
1,750,000 "	6,649	7,315	10,962	6,990	7,726	11,587	7,157	7,915	11,725	7,329	8,095	11,971
2,000,000 "	7,504	8,208	12,030	7,836	8,616	12,537	8,009	8,775	12,951	8,224	9,036	13,119

TABLE LXX

DIMENSIONS: VARNISHED-CAMBRIC INSULATED WIRES AND CABLES

Table shows approximate diameter of cambric insulated double-braided, flameproof, and single-braided, lead-covered, single-conductor stranded wires and cables. Thickness of braid 1/32 in. Voltage given is normal working voltage. Test voltage 2.5 times normal for 5 minutes.

Size	1,000 Volts				2,000 Volts				3,000 Volts				5,000 Volts			
	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead
6	.43	.47	1/16	5/64	.50	.53	3/32	5/64	.59	.62	9/64	5/64	.68	.75	3/16	1/16
4	.48	.51	1/16	5/64	.54	.58	3/32	5/64	.64	.70	9/64	1/16	.73	.76	3/16	1/16
3	.51	.54	1/16	5/64	.57	.60	3/32	5/64	.67	.73	9/64	1/16	.76	.82	3/16	1/16
2	.54	.60	1/16	1/16	.60	.67	3/32	1/16	.70	.76	9/64	1/16	.79	.89	3/16	5/64
1	.61	.68	5/64	1/16	.64	.71	3/32	1/16	.74	.80	9/64	1/16	.83	.93	3/16	5/64
0	.66	.72	5/64	1/16	.69	.75	3/32	1/16	.78	.87	9/64	5/64	.89	.97	3/16	5/64
00	.70	.76	5/64	1/16	.73	.79	3/32	1/16	.83	.92	9/64	5/64	.92	1.01	3/16	5/64
000	.75	.84	5/64	1/16	.78	.88	3/32	5/64	.87	.97	9/64	5/64	.97	1.10	3/16	3/32
0000	.81	.90	5/64	5/64	.84	.93	3/32	5/64	.93	1.07	9/64	3/32	1.03	1.15	3/16	3/32
250,000 C. M.	.89	1.01	3/32	3/32	.92	1.04	7/64	3/32	.98	1.11	9/64	3/32	1.03	1.20	3/16	3/32
300,000 "	.95	1.07	3/32	3/32	.98	1.10	7/64	3/32	1.01	1.17	9/64	3/32	1.13	1.26	3/16	3/32
350,000 "	.99	1.12	3/32	3/32	1.03	1.15	7/64	3/32	1.09	1.21	9/64	3/32	1.18	1.31	3/16	3/32
400,000 "	1.04	1.17	3/32	3/32	1.07	1.20	7/64	3/32	1.13	1.26	9/64	3/32	1.23	1.35	3/16	3/32
450,000 "	1.09	1.21	3/32	3/32	1.12	1.26	7/64	3/32	1.18	1.31	9/64	3/32	1.28	1.40	3/16	3/32
500,000 "	1.13	1.26	3/32	3/32	1.16	1.28	7/64	3/32	1.22	1.35	9/64	3/32	1.32	1.44	3/16	3/32
550,000 "	1.20	1.32	3/32	3/32	1.20	1.32	7/64	3/32	1.26	1.39	9/64	3/32	1.36	1.48	3/16	3/32
600,000 "	1.24	1.36	7/64	3/32	1.27	1.39	1/8	3/32	1.30	1.42	9/64	3/32	1.39	1.52	3/16	3/32
650,000 "	1.27	1.40	7/64	3/32	1.30	1.43	1/8	3/32	1.34	1.46	9/64	3/32	1.43	1.55	3/16	3/32
700,000 "	1.31	1.43	7/64	3/32	1.34	1.46	1/8	3/32	1.37	1.50	9/64	3/32	1.46	1.59	3/16	3/32
750,000 "	1.34	1.47	7/64	3/32	1.37	1.50	1/8	3/32	1.40	1.52	9/64	3/32	1.50	1.66	3/16	7/64
800,000 "	1.38	1.50	7/64	3/32	1.41	1.53	1/8	3/32	1.44	1.56	9/64	3/32	1.53	1.69	3/16	7/64
900,000 "	1.44	1.56	7/64	3/32	1.47	1.59	1/8	3/32	1.50	1.66	9/64	7/64	1.59	1.78	3/16	1/8
1,000,000 "	1.50	1.68	7/64	3/32	1.53	1.65	1/8	3/32	1.56	1.72	9/64	7/64	1.65	1.84	3/16	1/8
1,250,000 "	1.67	1.82	1/8	7/64	1.67	1.82	1/8	7/64	1.70	1.89	9/64	1/8	1.79	1.98	3/16	1/8
1,500,000 "	1.79	1.97	1/8	1/8	1.79	1.97	1/8	1/8	1.82	2.00	9/64	1/8	1.91	2.10	3/16	1/8
1,750,000 "	1.90	2.09	1/8	1/8	1.90	2.09	1/8	1/8	1.93	2.12	9/64	1/8	2.03	2.21	3/16	1/8
2,000,000 "	2.01	2.19	1/8	1/8	2.01	2.19	1/8	1/8	2.04	2.23	9/64	1/8	2.13	2.32	3/16	1/8

TABLE LXX—*Continued*

Size	7,000 Volts				10,000 Volts				13,000 Volts				17,000 Volts			
	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead	Braided	Leaded	Insulation	Lead
6	.81	.87	1/4	1/16	.93	1.03	5/16	5/64	1.06	1.15	3/8	5/64	1.18	1.31	7/16	3/32
4	.86	.95	1/4	5/64	.98	1.08	5/16	5/64	1.01	1.20	3/8	5/64	1.23	1.36	7/16	3/32
3	.89	.98	1/4	5/64	1.01	1.10	5/16	5/64	1.14	1.26	3/8	3/32	1.26	1.39	7/16	3/32
2	.92	1.01	1/4	5/64	1.04	1.14	5/16	5/64	1.17	1.29	3/8	3/32	1.29	1.42	7/16	3/32
1	.96	1.05	1/4	5/64	1.08	1.21	5/16	3/32	1.21	1.33	3/8	3/32	1.33	1.46	7/16	3/32
0	1.00	1.13	1/4	3/32	1.13	1.25	5/16	3/32	1.25	1.38	3/8	3/32	1.38	1.50	7/16	3/32
00	1.04	1.17	1/4	3/32	1.17	1.29	5/16	3/32	1.29	1.42	3/8	3/32	1.42	1.54	7/16	3/32
000	1.10	1.22	1/4	3/32	1.22	1.35	5/16	3/32	1.35	1.47	3/8	3/32	1.47	1.60	7/16	3/32
0000	1.15	1.28	1/4	3/32	1.28	1.40	5/16	3/32	1.40	1.53	3/8	3/32	1.53	1.65	7/16	3/32
250,000 C. M.	1.20	1.33	1/4	3/32	1.32	1.45	5/16	3/32	1.45	1.58	3/8	3/32	1.58	1.70	7/16	3/32
300,000 "	1.26	1.38	1/4	3/32	1.38	1.51	5/16	3/32	1.51	1.63	3/8	3/32	1.63	1.79	7/16	7/64
350,000 "	1.31	1.43	1/4	3/32	1.43	1.56	5/16	3/32	1.56	1.68	3/8	3/32	1.68	1.84	7/16	7/64
400,000 "	1.35	1.48	1/4	3/32	1.48	1.60	5/16	3/32	1.60	1.76	3/8	7/64	1.73	1.92	7/16	1/8
450,000 "	1.40	1.53	1/4	3/32	1.53	1.65	5/16	3/32	1.65	1.81	3/8	7/64	1.78	1.96	7/16	1/8
500,000 "	1.44	1.57	1/4	3/32	1.57	1.69	5/16	3/32	1.69	1.88	3/8	1/8	1.82	2.00	7/16	1/8
550,000 "	1.48	1.61	1/4	3/32	1.61	1.76	5/16	7/64	1.73	1.92	3/8	1/8	1.86	2.04	7/16	1/8
600,000 "	1.51	1.64	1/4	3/32	1 64	1.80	5/16	7/64	1.77	1.96	3/8	1/8	1.89	2.08	7/16	1/8
650,000 "	1.55	1.71	1/4	7/64	1.68	1.87	5/16	1/8	1.80	1.99	3/8	1/8	1.93	2.12	7/16	1/8
700,000 "	1.59	1.78	1/4	1/8	1.71	1.90	5/16	1/8	1.84	2.03	3/8	1/8	1.96	2.15	7/16	1/8
750,000 "	1.62	1.81	1/4	1/8	1.75	1.94	5/16	1/8	1.87	2.06	3/8	1/8	2.00	2.19	7/16	1/8
800,000 "	1.66	1.84	1/4	1/8	1.78	1.97	5/16	1/8	1.93	2.09	3/8	1/8	2.03	2.22	7/16	1/8
900,000 "	1.72	1.91	1/4	1/8	1.84	2.03	5/16	1/8	1.97	2.16	3/8	1/8	2.09	2.28	7/16	1/8
1,000,000 "	1.78	1.97	1/4	1/8	1.90	2.09	5/16	1/8	2.03	2.22	3/8	1/8	2.15	2.34	7/16	1/8
1,250,000 "	1.92	2.10	1/4	1/8	2.04	2.23	5/16	1/8	2.17	2.35	3/8	1/8	2.29	2.48	7/16	1/8
1,500,000 "	2.04	2.22	1/4	1/8	2.16	2.35	5/16	1/8	2.29	2.47	3/8	1/8	2.41	2.60	7/16	1/8
1,750,000 "	2.15	2.33	1/4	1/8	2.28	2.46	5/16	1/8	2.40	2.59	3/8	1/8	2.53	2.71	7/16	1/8
2,000,000 "	2.26	2.45	1/4	1/8	2.38	2.57	5/16	1/8	2.51	2.69	3/8	1/8	2.63	2.82	7/16	1/8

TABLE LXXI

Weights: Varnished-cambric Insulated Wires and Cables

Table gives approximate weights in pounds per thousand feet for wires and cables.

Unit Weights Used

Standard stranded annealed copper........................566 lb. per cu. ft
Rubber (either N.E.C. or 30 per cent Para) ....110 lb. per cu. ft.
Varnished cambric (as laid on the cable).................. 71 lb. per cu. ft.
Weatherproof braid (as laid on the cable)................. 70 lb. per cu. ft.
Flameproof braid (as laid on the cable)...................210 lb. per cu. ft.
Lead—pure...710 lb. per cu. ft.

Notes

1. Weights based on standard stranded annealed copper.
2. Weights based on double braid for braided conductors.
3. Weights based on single braid or tape between insulation and lead for leaded conductors.
4. Weights will apply without appreciable error to solid conductor wires and cables.

Size	1,000 Volts			2,000 Volts			3,000 Volts			5,000 Volts		
	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead
6 B. & S.	140	210	451	162	246	533	205	312	569	247	369	916
4 "	197	273	521	122	314	624	265	381	862	311	441	1,049
3 "	239	323	568	264	364	701	310	432	942	357	495	1,220
2 "	286	378	847	314	422	864	360	490	996	415	561	1,394
1 "	363	467	959	377	491	999	425	563	1,100	487	641	1,537
0 "	438	554	1,073	461	591	1,118	509	655	1,446	578	746	1,660
00 "	531	653	1,205	554	692	1,253	616	784	1,634	675	851	1,796
000 "	653	783	1,433	669	815	1,606	727	879	1,782	797	981	2,121
0000 "	802	956	1,812	820	974	1,905	886	1,062	2,294	960	1,160	2,431
250,000 C. M.	950	1,018	2,154	965	1,133	2,289	1,016	1,200	2,461	1,105	1,319	2,687
300,000 "	1,123	1,299	2,515	1,147	1,331	2,537	1,186	1,378	2,670	1,275	1,497	2,974
350,000 "	1,289	1,473	2,758	1,320	1,520	2,820	1,352	1,552	2,946	1,445	1,675	3,221
400,000 "	1,461	1,653	3,004	1,487	1,693	3,225	1,535	1,757	3,082	1,620	1,858	3,467
450,000 "	1,623	1,823	3,278	1,649	1,863	3,426	1,696	1,918	3,475	1,797	2,049	3,724
500,000 "	1,791	2,005	3,514	1,811	2,033	3,623	1,867	2,105	3,673	1,966	2,226	3,957
550,000 "	1,983	2,205	3,779	1,983	2,213	3,868	2,034	2,272	3,892	2,142	2,410	4,212
600,000 "	2,144	2,374	3,976	2,160	2,398	4,089	2,200	2,446	4,085	2,308	2,584	4,416
650,000 "	2,312	2,550	4,194	2,339	2,585	4,338	2,382	2,678	4,373	2,480	2,764	4,627
700,000 "	2,470	2,716	4,429	2,500	2,752	4,535	2,532	2,800	4,613	2,645	2,937	4,906
750,000 "	2,641	2,893	4,675	2,672	2,932	4,746	2,704	2,980	4,761	2,816	3,114	5,349
800,000 "	2,803	3,071	4,911	2,838	3,114	4,943	2,870	3,154	5,009	2,978	3,284	5,676
900,000 "	3,136	3,420	5,333	3,167	3,457	5,342	3,196	3,488	5,743	3,323	3,651	6,547
1,000,000 "	3,458	3,750	5,787	3,488	3,786	5,773	3,529	3,837	6,220	3,668	3,998	6,963
1,250,000 "	4,309	4,639	7,099	4,298	4,620	6,867	4,333	4,663	7,698	4,465	4,817	8,053
1,500,000 "	5,106	5,458	8,586	5,107	5,459	7,922	5,146	5,498	8,636	5,282	5,658	9,053
1,750,000 "	5,929	6,305	9,638	5,918	6,294	8,869	5,958	6,334	9,685	6,105	6,503	10,054
2,000,000 "	6,730	7,128	10,626	6,706	7,096	9,869	6,757	7,155	10,800	6,925	7,347	11,157

TABLE LXXI—*Continued*

Size	7,000 Volts			10,000 Volts			13,000 Volts			17,000 Volts		
	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead	W. P. Braid	F. P. Braid	Lead
6 B. & S.	330	484	1,037	403	563	1,575	509	701	1,812	601	815	2,343
4 "	391	551	1,447	478	654	1,689	515	577	1,964	701	931	2,562
3 "	446	614	1,536	540	732	1,813	647	859	2,340	755	993	2,688
2 "	508	684	1,625	598	798	1,956	706	928	2,402	819	1,073	2,793
1 "	580	764	1,736	669	875	2,262	786	1,016	2,563	920	1,188	2,942
0 "	668	860	2,139	772	994	2,436	983	1,121	2,741	1,020	1,312	3,089
00 "	772	972	2,318	881	1,111	2,576	1,002	1,254	2,891	1,138	1,446	3,179
000 "	902	1,108	2,565	1,019	1,257	2,788	1,145	1,405	3,091	1,287	1,595	3,456
0000 "	1,065	1,287	2,800	1,182	1,428	3,028	1,315	1,583	3,345	1,458	1,788	3,763
250,000 C. M.	1,211	1,541	3,026	1,335	1,587	3,296	1,467	1,751	3,616	1,629	1,967	3,950
300,000 "	1,388	1,626	3,274	1,520	1,788	3,598	1,660	1,952	3,886	1,817	2,155	4,576
350,000 "	1,570	1,816	3,530	1,713	1,997	3,895	1,853	2,159	4,144	1,999	2,337	4,918
400,000 "	1,748	2,000	3,794	1,896	2,194	4,122	2,037	2,351	4,801	2,188	2,540	5,548
450,000 "	1,922	2,190	4,042	2,073	2,379	4,370	2,219	2,541	5,060	2,382	2,742	5,870
500,000 "	2,095	2,379	4,290	2,255	2,569	4,617	2,404	2,740	5,693	2,568	2,944	6,125
550,000 "	2,275	2,567	4,547	2,438	2,760	5,074	2,594	2,988	5,938	2,771	3,169	6,430
600,000 "	2,447	2,745	4,869	2,604	2,934	5,403	2,767	3,121	6,370	2,949	3,347	6,680
650,000 "	2,631	2,937	5,317	2,786	3,122	6,132	2,937	3,241	6,529	3,130	3,531	6,932
700,000 "	2,796	3,110	5,990	2,960	3,304	6,376	3,112	3,472	6,787	3,297	3,695	7,182
750,000 "	2,973	3,295	6,240	3,136	3,488	6,634	3,302	3,670	6,965	3,505	3,935	7,456
800,000 "	3,138	3,468	6,479	3,302	3,662	6,877	3,498	3,922	7,203	3,666	4,058	7,670
900,000 "	3,485	3,811	6,902	3,658	4,026	7,296	3,827	4,211	7,716	4,009	4,407	8,090
1,000,000 "	3,619	4,165	7,395	4,011	4,401	7,708	4,188	4,594	8,145	4,370	4,800	8,650
1,250,000 "	4,651	5,019	8,451	4,839	5,237	8,723	4,970	5,480	9,422	5,242	5,686	9,738
1,500,000 "	5,472	5,862	9,458	5,675	6,087	9,934	5,892	6,336	10,412	6,083	6,549	10,702
1,750,000 "	6,319	6,741	10,511	6,521	6,957	10,968	6,736	7,188	11,435	6,987	7,501	11,850
2,000,000 "	7,135	7,579	11,585	7,361	7,843	11,960	7,602	8,086	12,533	7,841	8,355	12,995

and smaller when carrying voltages of 600 volts and less. For higher voltages or where somewhat better quality is desired, 30 per cent or 40 per cent Para or Hevea rubber is sometimes used. Kerite wire, which is insulated with a vulcanized compound of rubber and other ingredients, is especially adapted to resisting moisture and heat. Varnished-cambric insulation of various thicknesses is used for practically all the important wiring of the station. When varnished-cambric cables are used in vertical runs the oils are likely

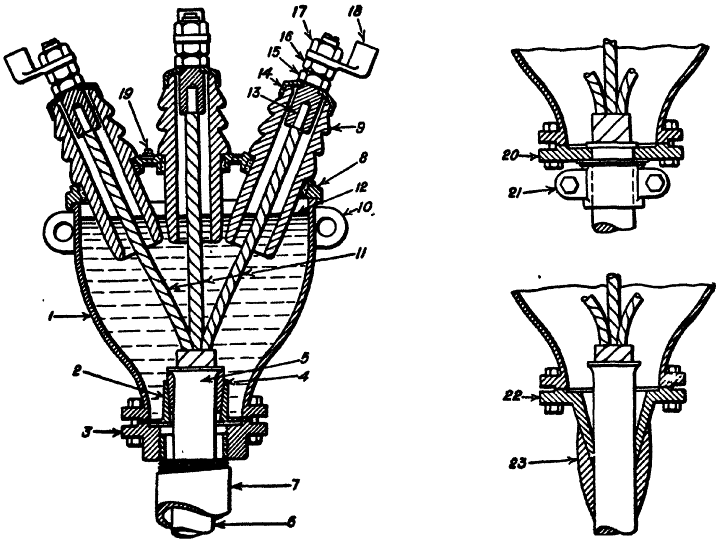


FIG. 455.—Cabel End Bell.

1. Bell Casing
2. Brass Bushing
3. Conduit Adapter or Companion Flange
4. Soft Metal Wedges
5. Cable Sheath
6. Lead Covered Cable
7. Conduit
8. Cover
9. Porcelain Bushings
10. Mounting Lugs
11. Conductors
12. Insulating Compound
13. Terminal Stud
14. Cementing Cap
15. Recessed Cap Nut
16. Contact Nut
17. Contact Nut
18. Terminal Lug
19. Filling Holes
20. Clamp Flange
21. Clamp Joint
22. Wiping Flange
23. Wipe Joint

to run out, but much trouble can be avoided from this source if the manufacturer is instructed to use suitable ingredients to meet this condition. Oiled-paper insulation costs less than either rubber or varnished cambric, but cables so insulated are likely to be injured by being bent sharply. For this reason, paper-insulated cables are seldom used in power-station conduits but are almost universally used for underground duct work. Tables LXVIII, LXIX, LXX and LXXI give approximate sizes, weights, and other data applying to standard insulated wires and cables. For the excitation system,

it is customary to use a minimum insulation thickness corresponding to a working pressure of 2000 volts. For three-conductor cables the conductor insulation and the belt insulation are usually each equal to half the insulation given for a single-conductor cable of the same working voltage; but in special cases, where the neutral of the three-phase system is solidly grounded, the belt insulation may be slightly reduced if desired.

Braids, or tapes and braids, are placed over the insulation to protect it from mechanical injury. For conduit work a weatherproof braid is used, and for exposed work a flameproof braid. For lighting circuits it is convenient to use white braid for the neutral wire to distinguish it from the other wires. For control, instrument, and signal wires it is sometimes found convenient to make use of several colors of braids for identification.

Lead sheaths are used in general where wires or cables are exposed to moisture, oil, or other agents which would cause the insulation to deteriorate. Single-conductor, lead-covered cables should be avoided for alternating-current circuits. If their use is found necessary, they should have their sheaths grounded thoroughly at both ends and should be designed for the extra heating caused by the sheath currents. End bells or potheads should always be used on lead-covered cables for voltages of 2000 volts or more. A typical cable end bell is shown in Fig. 455. For further data on underground cables see D. M. Simons, *Electrical Journal*, August, 1925, page 366.

The current-carrying capacity of a cable, where voltage drop is not the determining factor, is limited by the temperature which is considered safe for the insulation employed. The maximum safe limiting temperature at the surface of the conductor, as recommended by the Standards of the A.I.E.E., in degrees Centigrade, where E is the working voltage between conductors in r.m.s. kilo-volts is as follows:

For impregnated-paper insulation..................	$85-E$
For varnished-cambric insulation..................	$75-E$
For rubber insulation............................	$65-0.25E$

The ratings given by the National Electrical Code, Table LXXII, are based on somewhat lower temperatures than the above, and are, therefore, sufficiently conservative for the smaller circuits of the station wiring. For the more important circuits, however, and particularly for the higher voltages, they should be modified according to conditions as shown in Table LXXII.

To avoid injury to the insulation, a cable should not be bent to a radius of less than six times its outside diameter if rubber-insulated, seven times if cambric-insulated or sixteen times if paper-insulated. Rubber-insulated wires of less than ½-in. diameter, however, may be bent to smaller radii without injury.

Splices in cables are of great importance and deserve expert attention in order that the splices may be at least equal to any other part of the cable in conductivity, insulation, and protective covering. The essential features of the conductor connection are (1) carefully cut, cleaned, and tinned ends of the strands, (2) sleeve of correct size and design, preferably tapered, slotted

TABLE LXXII

CARRYING CAPACITIES OF WIRES AND CABLES
(From National Electrical Code, 1923 Edition)

B. & S. Gage	Diameter of Solid Wires, Mils	Area in Circular Mils	Rubber Insulation, Amperes	Varnished-cloth Insulation, Amperes
14	64.1	4,107	15	18
12	80.8	6,530	20	25
10	101.9	10,380	25	30
8	128.5	16,510	35	40
6	162.0	26,250	50	60
5	181.9	33,100	55	65
4	204.3	41,740	70	85
3	229.4	52,630	80	95
2	257.6	66,370	90	110
1	289.3	83,690	100	120
0	325.0	105,500	125	150
00	364.8	133,100	150	180
000	409.6	167,800	175	210
		200,000	200	240
0000	460	211,600	225	270
		250,000	250	300
		300,000	275	330
		350,000	300	360
		400,000	325	390
		500,000	400	480
		600,000	450	540
		700,000	500	600
		800,000	550	660
		900,000	600	720
		1,000,000	650	780
		1,100,000	690	830
		1,200,000	730	880
		1,300,000	770	920
		1,400,000	810	970
		1,500,000	850	1020
		1,600,000	890	1070
		1,700,000	930	1120
		1,800,000	970	1160
		1,900,000	1010	1210
		2,000,000	1050	1260

Working voltage:

	Per cent Amperes
2,300 volts	100
6,600 volts	95
13,200 volts	90

Number cables in adjacent ducts:

	Per cent Amperes
4 cables	100
6 cables	89
8 cables	80
10 cables	73
12 cables	67
16 cables	60
20 cables	55

Duration of operation:

	Per cent Amperes
Continuous	100
2 hours	105

Number conductors per cable:

	Per cent Amperes
Single-conductor	100
2-conductor parallel	87
2-conductor concentric	79
3-conductor triangular	75
3-conductor concentric	60

Ambient temperature of air:

	Per cent Amperes
90° F., 32° C.	100
100° F., 38° C.	92
110° F., 43° C.	82
120° F., 49° C.	71
130° F., 54° C.	58
140° F., 60° C.	40
150° F., 66° C.	0

Duration of Operation:

	Per cent Amperes
1 hour	125

and tinned, (3) careful butting of strands within the sleeve and thorough soldering to assure the entire sleeve and interstices between strands being completely filled with solder, and (4) careful inspection of finished connection and removal of all rough places or abrupt edges and of all filings and drops of solder. The essential features of the insulating are (1) careful penciling of original insulation avoiding loosening of layers, (2) filling of spaces at each end of sleeve with impregnated cotton tape or twine to provide a smooth surface for the taping, (3) careful taping with insulating tape similar to that of the cable, (4) saturating the tape with insulating compound as it is applied, (5) complete elimination of all voids or air pockets, (6) avoidance of all points, edges and abrupt changes in potential gradient, and (7) complete elimination of all moisture. The essential features of the protecting covering of the lead-covered cable splice are (1) a lead sleeve of ample size, properly tinned and provided with a pouring hole and a vent hole, (2) careful arrangement of taped joints within the sleeve, preferably separated from each other, (3) thorough soldering of sleeve to sheath, (4) careful selection of proper compound for particular service involved, (5) pouring of sleeve with hot compound so as to avoid froth or bubbles or voids from shrinkage and, (6) final closing of pouring and vent holes by soldering, and attaching tag bearing splicer's initials.

Fireproofing of certain exposed cables is sometimes advisable, as, for instance, at the main generator terminals. The essential features of such work are (1) filling up the irregularities at bolted joints with a paste of mica dust and insulating varnish to form a smooth surface for taping, (2) stripping back the flameproof covering on the cable and tapering the insulation, (3) carefully taping the joint with insulating tape saturated with insulating compound, (4) taping joint and adjacent cable with two layers of woven asbestos listing, and (5) impregnating and painting the asbestos listing with fireproof paint.

414. Conduits and Ducts.—Rigid iron conduit is used to enclose the greater part of the station wiring. It is manufactured from steel tubing, the interior being lined with flexible, insulating enamel, the outside either enameled, sherardized, electro-galvanized or hot-galvanized. It is made in 10-ft. lengths with each length reamed and threaded at each end and provided with one coupling. Many kinds of conduit fittings and devices are available. Good quality in both the conduit and the fittings can be assured by specifying that they shall meet the requirements of the Underwriters' Laboratories. Weights and dimensions of rigid iron conduits and the principal fittings are given in Table LXXIII.

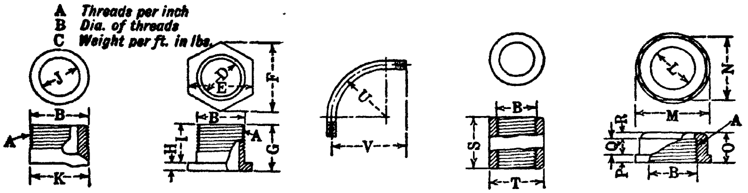
Installation of the conduit must keep pace with the building structure and, therefore, accurate and well-detailed conduit plans are of great importance. Galvanized or sherardized conduit is preferable for exposed work, especially in damp places, but for concealed work there is little choice between this and the black-enameled conduit. When conduits are installed in cinder concrete the cinders should be carefully selected to avoid acidity and should be thoroughly washed. It is also advisable to protect the conduits by asphalt paint or cement grout. The conduit system should be thoroughly grounded and other requirements of the National Electrical Code should be followed.

TABLE LXXIII

WEIGHTS AND DIMENSIONS: RIGID IRON CONDUIT AND FITTINGS

Dimensions—Inches

Conduit Nipples Elbows Couplings Bushings



Size of Conduit	A	B	C	D	E	F	G	H	I	J	K
½"	14.0	.82	.85	.62	1.00	1.15	.62	.12	.50	.62	.84
¾"	14.0	1.02	1.13	.82	1.25	1.44	.81	.19	.62	.82	1.05
1"	11.5	1.28	1.68	1.04	1.37	1.59	.94	.25	.69	1.04	1.31
1¼"	11.5	1.63	2.28	1.38	1.75	2.02	1.06	.25	.81	1.38	1.66
1½"	11.5	1.87	2.73	1.61	2.00	2.31	1.12	.31	.81	1.61	1.90
2"	11.5	2.34	3.68	2.06	2.50	2.89	1.31	.31	1.00	2.06	2.37
2½"	8.0	2.82	5.82	2.46	3.00	3.46	1.44	.37	1.06	2.46	2.87
3"	8.0	3.44	7.62	3.06	3.75	4.33	1 50	.37	1.12	3.06	3.50
3½"	8.0	3.94	9.20	3.54	4.25	4.91	1.62	.44	1.19	3.54	4.00

Size of Conduit	L	M	N	O	P	Q	R	S	T	U	V
½"	.62	1.00	.94	.37	.12	.19	.06	1.37	1.12	4.25	7.38
¾"	.75	1.25	1.12	.44	.12	.25	.06	1.56	1.31	5.38	8.38
1"	1.00	1.50	1.37	.50	.16	.25	.09	1.75	1.62	5.75	9.50
1¼"	1.25	1.81	1.75	.56	.19	.28	.09	2.12	2.00	7.25	10.88
1½"	1.50	2.12	2.00	.56	.19	.28	.09	2.50	2.25	8.25	12.63
2"	1.94	2.56	2.37	.62	.19	.31	.12	2 62	2.75	9.50	15.25
2½"	2.37	3.06	2.87	.75	.25	.37	.12	2.87	3.31	10.50	17.38
3"	2.87	3.75	3.50	.81	.25	.37	.19	3.06	3.93	13.00	19.50
3½"	3.25	4.25	4.00	1.00	.37	.44	.19	3.62	4.43	15.00	21.25

Weight—Pounds

Size of Conduit	Quantity in Feet						
	500	1,000	2,000	3,000	4,000	5,000	6,000
½"	426	852	1,704	2,556	3,408	4,260	5,112
¾"	567	1134	2,268	3,402	4,536	5,670	6,804
1"	842	1684	3,368	5,052	6,736	8,420	10,104
1¼"	1141	2281	4,562	6,843	9,124	11,405	13,686
1½"	1366	2731	5,462	8,193	10,924	13,655	16,386
2"	1839	3678	7,356	11,034	14,712	18,390	22,068
2½"	2910	5819	11,638	17,457	23,276	29,095	34,914
3"	3808	7616	15,232	22,848	30,464	38,080	45,696
3½"	4601	9202	18,404	27,606	36,808	46,010	55,212

Size of Conduit	Quantity in Feet						
	7,000	8,000	9,000	10,000	15,000	20,000	25,000
½"	5,964	6,816	7,668	8,520	12,780	17,040	21,300
¾"	7,938	9,072	10,206	11,340	17,010	22,680	28,350
1"	11,788	13,472	15,156	16,840	25,260	33,680	42,100
1¼"	15,967	18,248	20,529	22,810	34,215	45,620	57,025
1½"	19,117	21,848	24,579	27,310	40,965	54,620	68,275
2"	25,746	29,424	33,102	36,780	55,170	73,560	91,950
2½"	40,733	46,552	52,371	58,190	87,285	116,380	145,475
3"	53,312	60,928	68,544	76,160	114,240	152,320	190,400
3½"	64,414	73,616	82,818	92,020	138,030	184,040	230,050

Suitable conduit sizes to be used for various groupings of wires and cables are given in Table LXXIV. The conduits should be carefully cleaned and dried before the wires are pulled in.

TABLE LXXIV

Suitable Conduit Sizes for Various Groupings of Wires and Cables

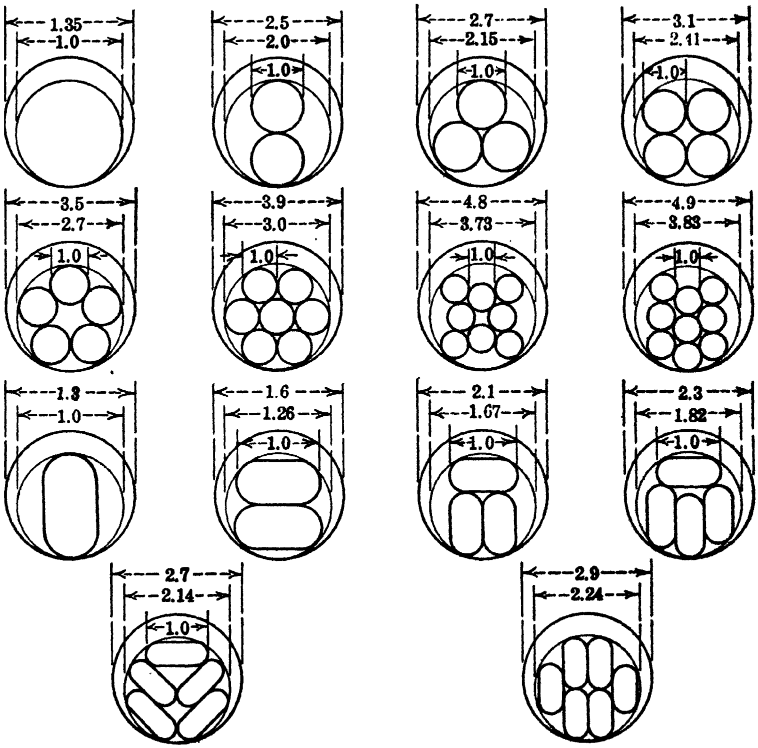
Diagram shows smallest equivalent diameter of group of wires and diameter of conduit in terms of diameter of a single wire.

Diameter of conduit is for runs of from 50 ft. with two-90° bends, to 150 ft. with one-90° bend.

For more difficult runs and in all cases where cables are larger than one inch in diameter, increase diameter of conduit by 15%.

Example: Size of conduit for four- No. 0 wires. Diameter of No. 0 wire = 0.63

$$0.63 \times 3.1 = 1.953 = 2'' \text{ conduit.}$$



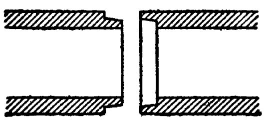
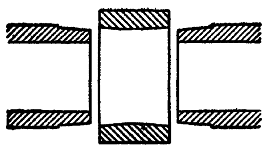
Internal Diameters of Iron Conduit				
½″	¾″	1″	1¼″	1½″
0.623	0.824	1.05	1.38	1.61
2″	2½″	3″	3½″	4″
2.07	2.47	3.07	3.55	4.03

Metallic conduits other than rigid iron are often of great value in power-station construction. Flexible steel conduit is used for connections at motors and machines. Aluminum or brass conduits are often needed for large conductors carrying alternating current, on account of their non-magnetic qualities.

Fiber conduit is made by wrapping thin wood pulp on mandrels and then impregnating with pitch. Standard sizes are shown in Table LXXV. Smaller and larger sizes than those shown can be obtained by special order. Shorter-radius bends than those shown can also be obtained if required. The available fittings include couplings (for the Harrington joint conduit), bends, and offsets. On account of its comparative fragility, fiber conduit should always

TABLE LXXV

ORANGEBURG FIBER CONDUIT

Type of Joint	Inside Diameter, Inches	Net Weight, Pounds per Foot	Radius Standard 45° and 90° Bends, Inches
Socket 	2	0.92	18–24–36
	2½	1.10	24–36
	3	1.35	36
	3½	1.65	36
	4	1.85	36
	4½	2.25	36
Harrington 	2	0.95	18–24–36
	2½	1.20	24–36
	3	1.45	36
	3½	1.75	36
	4	2.00	36
	4½	2.45	36

Wall thickness, approximately ¼ in.
Standard length, 5 ft.

be enclosed in concrete. Before the concrete is poured, all joints should be wrapped in burlap and painted with hot tar. Fiber conduit is lighter and less expensive than iron conduit and is, therefore, used in preference to iron quite generally for concealed work in sizes of 2 in. and larger.

Tile duct is made in single sections 18 in. long and multiple sections 36 in. long. It is seldom used in the power station, but is often used in underground duct lines. Split tile is sometimes advantageous in cable rooms and manholes, as it can be placed around the cables after they are in place. Multiple-tile duct is not suitable for installing heavy capacity circuits as the fragile partitions are easily destroyed by a cable burn-out. For heavy-capacity duct

lines, each duct, whether of tile or fiber, should be surrounded by at least 3 in. of concrete.

415. Copper-bar and Tube Specification.—The following brief outline is intended as a guide to the principal items to be covered in a specification for copper bar or tubing for use as electrical conductors:

General

Quantity wanted; reference to latest Standards of the A.I.E.E.; for use as electrical conductors; to be furnished in standard mill lengths of 12 to 20 ft.; inspection; manufactured by rolled or drawn process.

Purity

Not less than 99.88 per cent for bar or 99.0 per cent for tube as determined by electrolytic assay, silver being counted as copper.

Conductivity

Not less than 97 per cent for bar or 85 per cent for tube, of the A.I.E.E. Standard for annealed copper.

Bending Tests

Shall be capable of being bent cold to a right angle in either direction to the following inside radii, without development of cracks:

Thickness of Bar, Inches	Inside Radius Inches	Diameter of Tube, Inches	Inside Radius, Inches
$\frac{1}{16}$	$\frac{1}{16}$	$\frac{1}{2}$	4
$\frac{1}{8}$	$\frac{1}{8}$	$\frac{3}{4}$	$5\frac{1}{4}$
$\frac{1}{4}$	$\frac{5}{16}$	1	$6\frac{1}{4}$
$\frac{3}{8}$	$\frac{9}{16}$	$1\frac{1}{4}$	$7\frac{1}{4}$
$\frac{1}{2}$	$\frac{3}{4}$	$1\frac{1}{2}$	8
		2	$9\frac{1}{2}$

Rigidity

Shall be as hard and rigid as is practicable and consistent with other requirements of this specification.

Shape and Finish

Bars shall be straight, of rectangular cross-section, with plain surfaces and of uniform width and thickness. Variation of surface of bar from a perfect plane along a length equal to the width of the bar shall not exceed 0.001 in., plus or minus. Variation of thickness of bar from nominal thickness shall not exceed 0.002 in., plus or minus. Variation of width of bar from nominal width shall not exceed 0.010 in., plus or minus.

Tubes shall be straight, of circular cross-section and of uniform outside

diameter and wall thickness. Variation of outside diameter from nominal outside diameter shall not exceed 0.003 in., plus or minus. Variation of wall thickness from nominal thickness shall not exceed 0.006 in., plus or minus.

416. Lightning Arresters.—A lightning arrester is a device for protecting circuits and apparatus against lightning and other abnormal potential rises of short duration. It is rated by the voltage of the circuit on which it is to be used. The ideal arrester would immediately relieve the system of excess voltage, allowing no flow of dynamic current, and at no time during or after this performance would it be unprepared for immediate service again. Arresters are of two general types: (1) the "gap and resistance" type wherein a series gap determines the voltage at which it will discharge and the discharge is proportional to the total applied voltage, and (2) the "valve" type wherein the chracteristics of the cells largely determine the voltage at which it will discharge, and the discharge is proportional to the excess voltage above a certain critical value.

Of the "gap and resistance" type, the simplest and least expensive form consists of a single gap in series with a resistance between line and ground. It is used at all voltages, either a.c. or d.c. At normal frequency the resistance has little effect, but at impulse frequency it reduces the speed of the gap by impeding the charging current. After the gap breaks down the resistance limits the current flow, thus helping to extinguish the arc at the gap. The current flow is proportional to the total applied voltage. A high resistance allows closer gap setting and a low resistance allows a more effective discharge. The choice must be a compromise, which results in a sacrifice in efficiency. Another type has a gap, resistance, and magnetic blow-out. A lower resistance and closer gap setting can be used, and this type is quite satisfactory, especially for d.c. service. A third type consists of a gap and a fuse in series. It is effective but non-restoring. Another of this type consists of a gap, resistance, and self-restoring circuit-breaker in series. It is fairly satisfactory for d.c. service, but has the disadvantages that its operation depends upon moving parts and that, while the breaker is restoring itself, the arrester is unprepared for duty. The Bennett arrester consists of a single gap in series with a water-column resistance. The current enters the water column at the top through a carbon electrode and the heat vaporizes a portion of the water, driving part of the water out through an opening in the bottom into another chamber, thereby increasing the resistance and interrupting the arc. The arrester is adaptable to all voltages and to a.c. or d.c. service. Another single-gap arrester, known as the Multi-Path or Carborundum arrester, consists of a single gap in series with a block of carborundum. The resistivity and structure of the carborundum cause the discharge to spread over the entire block, avoiding concentrated heating and consequent arcing. It is used on a.c. or d.c. service up to 750 volts. The condenser type arrester consists of an electrostatic condenser with or without a series gap. It is used for d.c. service only, up to 1500 volts.

The Multigap arrester, which is also a "gap and resistance" type, consists of a series of small gaps in combination with resistance. The gaps occur between a series of relatively large cylinders of non-arcing metal, usually a

brass rich in zinc. Owing to condenser action of the cylinders the impressed potential is not distributed uniformly over the gaps, but the gradient is very steep at the line end and gradual at the ground end. The gradient at the line end becomes even more steep at lightning frequency. Under discharge, however, the gradient is practically uniform from end to end and all gaps are effective in interrupting the discharge. The Low Equivalent arrester is a multigap arrester consisting of multigaps in series with a resistance. It is used for a.c. service up to 39,000 volts. The Compression arrester consists of multigaps in series with a resistance, the gaps being enclosed in air-tight chambers. The Graded Shunt Resistance arrester consists of multigaps bridged by resistors of various values connected between the line end and various points along the series of cylinders. By thus distributing the impressed voltage, the current can be discharged more quickly and successfully interrupted.

Of the " valve " type arresters, the Aluminum Cell or Electrolytic is probably the most extensively used and best known, although it has recently been superseded for a.c. service by other equally efficient types. The electrolytic arrester takes practically no current at normal voltage, discharges vigorously at relatively slight excess voltage, and immediately restores itself. Essentially it consists of a stack of aluminum trays, nested, insulated from each other, partially filled with electrolyte, and finally immersed in a steel tank of oil, thus forming a series of electrolytic cells between line and ground. For a.c. service a series gap is used to prevent the slight charging current that would result from the condenser action of the cells, but for d.c. service the gap is not required. With the application of rated potential, which is about 250 to 300 volts per cell, a film of aluminum hydroxide is formed on each aluminum surface. The film effectively prevents current flow at the voltage at which it was formed, but at temporarily higher voltage it allows a discharge in proportion to the excess voltage. As the film gradually dissolves, when gaps are used so that line potential is not constantly applied, arresters on a.c. service must be charged daily by closing the gaps for a few seconds. This requirement, and the fire hazard from the oil, constitute the principal objections to this type of arrester.

The Oxide Film arrester is another form of " valve " arrester adaptable to all voltages, a.c. and d.c., and is offered by one manufacturer in preference to the aluminum arrester. It consists essentially of a stack of cells in series with a gap. Each cell consists of two brass disks separated by an annular ring of poreclain, the interior space being compactly filled with lead peroxide powder which has a low resistance. A film of varnish on the side of each brass disk in contact with the lead peroxide provides sufficient insulation to prevent appreciable current flow at rated voltage, which is about 300 volts per cell. A series gap is used to prevent the very slight leakage current that would otherwise flow at rated voltage. At abnormal voltage the gap is broken down and the varnish films are punctured. The current flow is proportional to the excess voltage. The action of the current on the lead peroxide is to form red lead and litharge, which immediately seal up the punctures, restoring the insulating value of the original varnish film, and the arrester

is again ready for duty. The Pellet type arrester is a modification of the Oxide Film type and is particularly adaptable to distribution circuits. The lead peroxide is formed into small pellets which are coated with an insulating powder and placed in a porcelain chamber between two electrodes. In this way, each pellet, with its insulating powder covering, constitutes a miniature cell corresponding to the larger and less numerous cells of the Oxide Film arrester. A gap is used in series.

The Auto-valve arrester is another " valve " type arrester suitable to all voltages, a.c. and d.c. and is offered by another manufacturer in preference to the aluminum arrester. It consists of a stack of carborundum disks separated by very thin insulating washers. A gap is used in series. The discharge spreads over the entire surface of the carborundum in the form of a

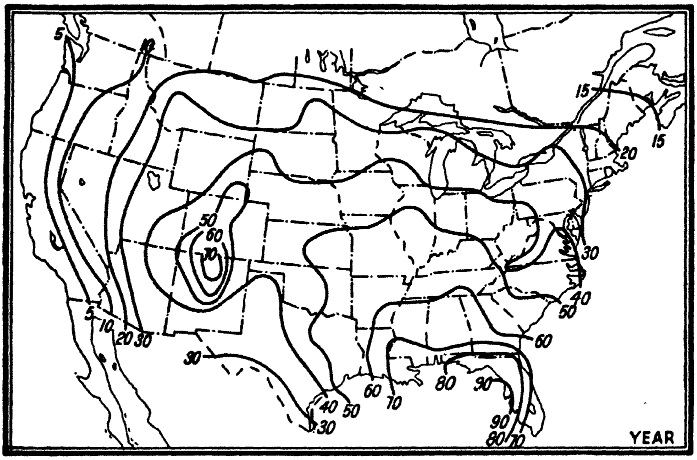


Fig. 456.—Isoceraunics, Year Based on Average Number of Thunderstorm Days, 20 Years; 1904–1923.

From "Monthly Weather Review," July, 1924, p. 340.

glow, thus preventing localized heating and consequent arcing. As no arc is formed the discharge is immediately interrupted when normal voltage is resumed.

The relative importance of lightning protection in various parts of the United States is shown on the map of Fig. 456.

417. Gaps for Arresters.—A gap is generally used between the arrester and the line to prevent discharge with moderate voltage rise and to prevent leakage of current through the arrester. Where the gap is used to interrupt the discharge, it is usually provided with horns to lengthen the arc as it spreads upward by its own heat.

To afford effective protection, a gap should discharge an impulse very rapidly. A needle gap is slow-acting as compared with a sphere gap, and therefore a sphere gap, or at least a gap having large surfaces compared to its

spacing, is always used. The sphere gap, when wet, discharges at about half the voltage required to make it discharge when dry. For this reason the sphere gap for outdoor use is usually covered to keep it dry, as it can then be set for a smaller margin above normal voltage.

A modification of the sphere gap, known as the Impulse gap, utilizes an assembly of two condensers and a resistance bridged across the gap for the purpose of obtaining greater sensitiveness to impulse frequencies.

Another modification of the sphere gap, known as the Compensated gap, consists of small spheres surrounded by large disks of refractory porcelain. When dry, the small spheres are effective, the porcelain being an insulator. When wet, the porcelain absorbs moisture so as to increase effectively the size of the spheres, and as a result the gap has about the same breakdown voltage whether wet or dry.

418. Choke Coils.—Choke coils are used in the main-line conductors between the apparatus to be protected and the point where the arrester is connected. The choke coil is essentially an air-core reactor. The impedance is small at normal frequency, but at impulse frequency it materially assists in directing the disturbance into the arrester. As the choke coil is in the main line and subject to system short-circuit currents, it must be mechanically braced to withstand the severe stresses.

419. Grounding.—The station grounding system performs three classes of service: (1) safeguarding of employees, (2) protection of circuits and apparatus, and (3) operation of relays. In all cases it is very important that a reliable, low-resistance, large ampere-capacity ground connection be installed. Much assistance in the proper location of grounds can be obtained from United States Geologic and other soil maps. Large buried bodies of steel or iron, such as penstocks, abandoned steel sheeting, and water mains, often form excellent grounds, but in some cases apparently well-grounded objects and structures are found by test to have a considerable resistance to ground. For instance, a copper plate thrown into the bed of a mountain stream is often found to be surprisingly insulated from earth on account of the purity of the water and the rock formation below. It is, therefore, always necessary to select carefully the location of the grounds and to test them thoroughly, not only when they are installed, but periodically during their service.

A copper-plate ground is made by burying a plate of copper about 3 or 4 ft. square sufficiently deep in the earth to lie in permanent moisture. Crushed coke or charcoal should be packed around the plate to a depth of about 2 ft. above and 2 ft. below, and the excavation should be filled with soil mixed with common salt. The ground wire should consist of stranded copper and should be well spread out over the plate and riveted and soldered.

The iron-pipe ground is very easily installed and very effective. Galvanized-steel, 1 to 1½ in. pipes, without joints, should be driven to permanent moisture. Suitable points and driving caps are available. The ground wire, which should consist of galvanized-steel stranded or of copper stranded cable, should be connected to the pipe by suitable clamps, or may be soldered by plugging the pipe about 12 in. below the top, inserting a loop of the cable and filling with a solder consisting of 75 per cent lead and 25 per cent antimony,

which will not contract as it cools. Pipe grounds should not be placed closer to each other than 6 ft. At least two pipes should be used for every installation, and many more are preferable. In some instances, it is advisable practically to surround the station with ground pipes. No rule can be given as to maximum resistance allowable, but each pipe, under average conditions, would be expected to have a resistance of from 10 to 25 ohms.

Permanent provision should be made for convenient routine tests of the ground resistance, and for this reason it is desirable to group the pipes and bring their terminals into the station through insulated cables to suitable test points before connecting them to the station ground bus. The method of testing ground resistance, known as the "Three-current Terminal Method," and described fully in Technologic Paper No. 108 of the Bureau of Standards entitled "Ground Connections for Electrical Systems," is recommended. The procedure is as follows:

1. Obtain a source of alternating-current supply at about 110 or 220 volts single-phase and reduce it by means of a two-winding transformer to such voltage as will give about 10 to 50 amperes through two of the ground pipes in series. A two-winding transformer is essential to secure a testing voltage that is positively ungrounded.
2. Select a voltmeter and an ammeter of proper scales.
3. Letter the ground pipes A, B, C, D, etc.
4. With one side of the test circuit connected to A and the other through the ammeter to B, read amperes through A and B and voltage across $A-B$. The volts divided by the amperes gives the resistance in ohms, R_{AB}, of pipes A and B.
5. Repeat for pipes B and C obtaining R_{BC} and for pipes C and A obtaining R_{CA}.
6. Compute resistances of A, B and C from equations:

$$\text{Resistance of } A = R_A = \frac{R_{AB} - R_{BC} + R_{CA}}{2}.$$

$$\text{Resistance of } B = R_B = \frac{R_{BC} - R_{CA} + R_{AB}}{2}$$

$$\text{Resistance of } C = R_C = \frac{R_{CA} - R_{AB} + R_{BC}}{2}.$$

7. Repeat entire process with pipes B, C and D, securing a check on R_B and R_C and new value R_D. By proceeding through all the pipes in this manner the last test should include A, giving a check on R_A.

The ground bus should be of copper bar bolted to the walls and structures. Ground conductors should never be run through metal pipes unless a good electrical connection is made between conductor and pipe at each end. For running a stranded ground cable through a concrete floor or wall, a very durable and attractive installation is obtained by running the cable through a brass pipe, using a reducing coupling at each end, the smaller end of each coupling being drilled out to fit the cable and securely sweated thereto.

Equipment to be grounded includes the following:

Switchboard frames and supports.

Instrument and meter cases.

Non-current-carrying metal parts of switchboard devices with which the attendant may come in contact.
Transformer frames and cases.
Oil circuit-breaker frames and mechanisms.
Rheostats.
Generators, motors and starters.
Instrument-transformer secondaries, at the transformers.
Lighting-transformer neutrals.
Steel supports and structures.
Bases of disconnecting switches and fuses, especially when mounted on masonry.
Lightning arresters. These should have individual grounds with straight, direct ground connections.

For further information on grounding of power systems, see "General Considerations in Grounding the Neutral of Power Systems," by H. H. Dewey, *Transactions* A.I.E.E., Vol. XLII, page 405, April, 1923.

420. Auxiliary Power and Lighting.—Power for the station auxiliaries and building services and lighting for the station, grounds, and works constitute very important parts of the design because they are necessary to support the production of outgoing energy. The advertising value of a well-lighted station is also often worthy of consideration. The total amount of power required as a percentage of the station output will vary from 0.5 per cent to 5.0 per cent, depending on size of station, general type of hydraulic development, and many other factors.

The source of supply for the auxiliary power and lighting is usually from the main low-voltage or high-voltage bus through transformers, although with the larger stations consideration should be given to auxiliary generators driven either by the main water wheels or by small individual wheels. Usually, if auxiliary generators are used, it is advisable to provide standby transformers.

The voltage is generally 2200 volts, three-phase, for motors of over 40 h.p.; 440 volts, three-phase for smaller motors; and 220–110 volts, single-phase, three-wire, for lighting. Where the main bus is used as a source of supply it is necessary to use feeder voltage regulators on the lighting, but when auxiliary generators are used the generator voltage regulators are sufficient. For scattered lights, as along the crest of the dam, it is often advisable to use series circuits.

421. Power Wiring and Equipment.—In providing for the auxiliary power load, the following items should be considered:

Excitation (see Sec. 378)
Turbine-room cranes
Intake-house cranes
Gantry cranes
Headgates
Screen racks
Sluice gates

Locks
Forebay ice prevention
Transmission-line ice prevention
Air compressors
Lubricating-oil pumps
Governor oil pumps
Transformer oil and water pumps
Filter presses
Ventilating fans
Generator-cooling fans
House service pumps
Sump pumps
Electric heating
Battery charging
Machine shop

A carefully designed double bus or sectionalized ring bus system should be used, and circuits should be duplicated to important motors so that the retirement of any part of the system by accident or for purposes of maintenance will not curtail the station output.

Motors and controllers should be carefully selected for the particular requirements of each service. Table LXXVI gives performance data of standard three-phase induction motors. The power factor is given at full load. At 75 per cent load the power factor is from 90 to 95 per cent of its value at full load, and at 50 per cent load it is from 70 to 80 per cent of its value at full load. The efficiency may be obtained from the relation:

$$\text{Amperes} = \frac{\text{Horse power} \times 746}{1.73 \times \text{volts} \times \text{power factor} \times \text{efficiency}} \quad \text{(209)}$$

A standard induction motor may be started by being thrown directly across the line without injury to the motor, but with sizes larger than 5 or 7½ h.p. the starting current is sometimes objectionable from the standpoint of the system voltage regulation. The double squirrel-cage motor is designed to take 25 to 50 per cent less starting current than a standard motor and is, therefore, very satisfactory for station auxiliary drive. Slip-ring motors are sometimes required where slow acceleration is desired. Push-button control for all motors is to be recommended as the starting operations are simple and uniform, and generally a more convenient arrangement of equipment can be obtained.

422. Induction-motor Specification.—The following condensed outline is intended as a guide to the principal items to be covered in an induction motor specification.

General

Number wanted; reference to latest Standards of the A.I.E.E.; material and workmanship; inspection; cooperation with other contractors.

TABLE LXXVI

Standard Induction-motor Data

Sizes: ¾, 1, 1½, 2, 3, 5, 7½, 10, 15, 20, 25, 30, 40, 50, 60, 75, 100, 125, 150, 200 h.p.

Horse Power Squirrel Cage	Voltages	Horse Power Slip Ring
½ to 2	110, 220, 440, 550	½ to 2
3 to 15	220, 440, 550	3 to 20
20 to 125	220, 440, 550, 2200	25 to 100
150 to 200	440, 550, 2200	125 to 200

Typical Full-load Performance, 60-cycle, Three-phase

(440-volt below 40 h.p.; 2200-volt above 40 h.p.)

Rated Horse-power	Syn. Speed	Squirrel Cage			Slip Ring			
		Rated Speed	Power Factor	Amperes	Rated Speed	Power Factor	Amperes Pri.	Amperes Sec.
1	1800	1710	80	1.5				
	1200	1130	75	1.7	1100	70	1.9	10.0
	900	855	70	1.8				
2	1800	1740	86	2.7	1700	82	3.0	17.2
	1200	1155	82	2.9	1115	72	3.5	18.0
	900	860	70	3.5	850	67	3.9	18.0
5	1800	1740	89	6.4	1700	86	6.8	30.5
	1200	1150	86	6.6	1140	81	7.4	27.3
	900	865	77	7.5	855	67	9.0	27.2
10	1800	1740	91	12.1	1725	86	13.0	28.0
	1200	1160	88	12.7	1145	84	13.7	30.0
	900	860	81	14.0	840	76	15.4	50.0
20	1800	1750	88	25.0	1720	87	26.0	61.0
	1200	1160	86	25.5	1135	84	27.5	82.0
	900	865	84	26.0	855	80	28.5	95.0
30	1200	1150	90	36.5	1130	88	38.5	106.0
	900	860	88	37.5	850	83	41.0	132.0
	720	685	86	39.0	690	82	41.0	81.0
50	1200	1150	88	12.5	1150	87	13.0	133.5
	900	870	85	13.0	865	83	13.5	116.0
	720	685	86	13.0	695	77	14.5	100.0
75	1200	1170	87	19.0	1170	86	19.2	81.0
	900	865	90	18.5	860	88	19.0	145.0
	720	690	85	19.5	685	82	20.5	136.0
100	900	865	90	24.3	870	89	25.0	138.0
	720	690	87	25.0	705	88	25.0	151.0
	600	575	83	26.0	580	83	26.5	125.0
150	900	870	91	36.0	880	89	37.0	129.0
	720	695	89	36.0	705	87	37.0	164.0
	600	580	88	37.0	580	88	37.0	196.0
200	720	700	89	49.0				
	600	580	88	50.0	585	88	49.0	195.0
	450	430	80	54.5	435	82	52.5	182.0

Rating

Horse power or kilowatt output; volts; cycles per second; synchronous speed; number of phases.

Type

Squirrel-cage; double squirrel-cage; slip-ring; constant or adjustable-speed; belted, geared or coupled; horizontal or vertical-shaft; open or enclosed-frame.

Service

Description of service motor is to perform, including name of machine to to be driven, atmospheric conditions if unusual, and starting requirements if unusual.

Temperature Rise

Temperature rise of stator and rotor after full load continuously; rise after overloads of one or two hours for certain types of service. Standard motors are designed for a rise not exceeding 40° C. above an ambient temperature of 40° C. after continuous operation, temperatures being measured by thermometer.

Arrangement

Location of terminals; conduit boxes; coupling or pulley; direction of rotation; number of bearings; adjustable base if for belt drive.

Special Features

Impregnation of windings to resist moisture; finish, if not standard; pin gage for fitting coupling, if latter is to be furnished by another manufacturer; shipment to another manufacturer for mounting.

Control

Starting equipment, such as line switch, starting switch, reversing switch, starting compensator, starting panel, instruments, meters, push buttons, float switch, pressure switch; speed-control equipment, such as secondary resistance, drum controller, carbon pile, giving various speeds desired and corresponding power outputs; protective equipment, such as relays, fuses, thermal cut-outs, overload, undervoltage, reverse phase.

Data to be Submitted by Bidder

Guaranteed efficiencies and power factors at 100, 75 and 50 per cent loads; guaranteed full-load speed; starting torque; pull-out torque; primary and secondary starting currents; primary and secondary full-load currents; manufacturer's designation of motor and of controller; general weights and dimensions.

Data to be Submitted by Contractor

Certified test reports verifying above guarantees; certified drawings for approval, for station design, and for erection.

423. Lighting Wiring and Equipment.—In providing for the station lighting load the following items should be considered:

Power station
Outdoor switch yard
Yard and grounds
Storehouses, machine shops
Dam
Gate house
Intake house
Screen racks
Locks
Employees' cottages and buildings
Display signs and flood lighting

A reliable source of uniform voltage is necessary as a supply for the lighting system and this is usually obtained through transformers from the auxiliary power bus. (See Sec. 420.) The distribution system should, for convenience, consist of a separate circuit from the switchboard to each cabinet, but for reasons of economy it is usually necessary to group several cabinets on each feeder. The neutral of the three-wire system should be grounded at the transformer and unfused throughout the system. The use of white braid for the neutral wire of the three-wire mains and of the two-wire branches is very convenient. Cabinets should generally be treated as fusing points only, and located to best economic advantage from this standpoint, the switching being done by local wall switches. For some purposes, however, cabinet branch switches are convenient. In the turbine room, for example, the entire cabinet is often controlled magnetically by a remote switch, but cabinet branch switches are used to reduce the illumination in times of normal operation and increase it in times of repairs or maintenance.

Emergency lighting is very essential. Three systems in common use are shown in Fig. 457. System C has the advantages that the emergency lamps burn on the battery only during times when the normal source is dead; that no duplication of lamps is required at gages and other important points; and that the emergency lamps form a part of the normal system, outages being quickly noticed and corrected. The battery should have capacity to supply the emergency lighting for a period of one hour.

Equipment, such as switches, receptacles, and lighting units, should be rugged, convenient to operate, and accessible for maintenance. A large number of carefully located receptacles are of great assistance, both during construction and for maintenance. Consideration should be given to the use of water-tight equipment in all places subject to moisture. Guards should be used on all lights in passageways as these are subject to breakage.

424. Illumination Design.—The usual methods used in designing industrial lighting are applicable to such portions of the power station as the

turbine room, switchboard room, offices, and similar open spaces. For the wheel pits, passageways, oil rooms, bus rooms, switch rooms, and similar parts of the building, however, careful attention should be given to side lighting, elimination of shadows, avoidance of glare, and individual location of lights with respect to machinery. It is often necessary, in such places, to use a large number of small lights advantageously located, rather than a few large lights as would be preferable in an open space.

Glare and sharp shadows can be reduced by the use of enclosing globes of good diffusing glass. Ordinary ground glass and many kinds of opalescent glass are entirely inadequate as diffusers. Brilliancy can be reduced by using oversize enclosing globes.

Metal guards should be used wherever lights are subject to mechanical injury. They are generally needed in the wheel pits, tunnels, and passages.

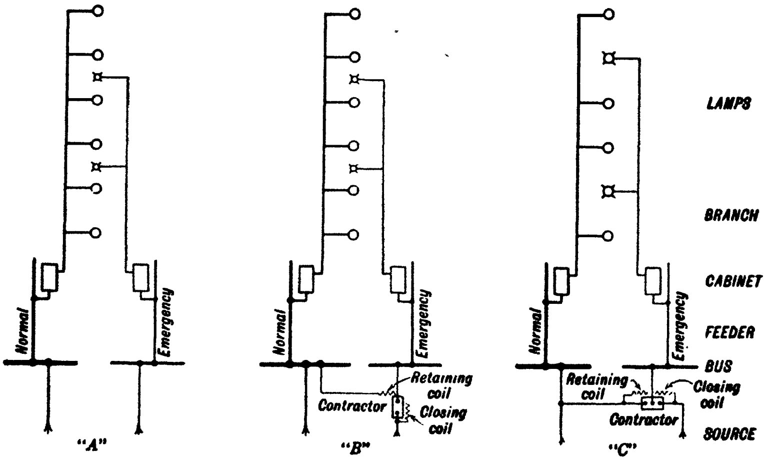


FIG. 457.—Diagrams of Emergency Distribution Systems.

A. Emergency lamps always burn on emergency source.
B. Emergency lamps normally dark, but in emergency burn on emergency source.
C. Lamps normally burn on normal source, but in emergency burn on emergency source.

Three-way switching is essential wherever long rooms or passages are frequently entered from either end. Bus rooms, switch rooms, tunnels, and passages usually require this type of switching.

The large interior, such as the turbine room and gate house, require high-capacity direct lighting units on the ceiling above the crane. Prismatic reflectors are suitable for this purpose as they efficiently direct most of the light to the working plane, and at the same time provide some illumination for the upper part of the room. In the turbine room, the switching should provide at least two intensities. If desired for decorative effect, wall lights can be used in addition to the overhead lights, but they should be of low brilliancy and should not be expected to contribute appreciably to the illumination intensity.

Oil rooms should have gas-proof fittings. Switches and receptacles should be located outside. Battery rooms should have acid-resisting fittings. The use of light-colored paint in the battery room greatly improves the lighting.

Bus rooms and switch rooms having barriers require careful design in order to light the interiors of the compartments. Sub-cell disconnecting switches and ceiling disconnecting switches require particular attention. Much can be accomplished by using white paint on the walls, ceiling, and barriers and by using a relatively large number of highly diffusing lighting units. Sometimes it is advisable to use wall lights.

The main switchboard room should have comfortable, attractive lighting. It should provide for operation of the controls, reading of instruments, clerical work at the desk, and repair work behind the panels. Overhead units of the semi-indirect type are suitable as they give a strong component of illumination on the vertical boards and minimize glare and shadows. They should be designed for distributing not over 10 per cent of their total output below the horizontal, and the ceiling should have a dull white finish. If these precautions are not regarded, disagreeable reflections are likely to occur on the faces of the instruments. A glass ceiling is sometimes provided in preference to windows in the walls. A skylight over the glass ceiling provides natural light by day, and lighting units between the ceiling and the skylight provide artificial lighting by night. The reflectors should be of glass if placed in the path of the natural light, but may be of steel if placed at the sides. Prismatic sheet glass in the ceiling can be used to direct the light toward the switchboard; but this feature is of small importance, and generally the more attractive rippled glass is preferred. Where a glass ceiling is used, it is very difficult to prevent reflections from instrument faces, and this feature should have careful consideration.

The fundamental relation between lamp wattage and illumination intensity for a given area or room is expressed by the equation:

$$W = \frac{IAD}{EL}, \qquad (210)$$

where, W = total wattage of lamps, in watts;

I = illumination intensity in foot-candles. Suitable values are given in Table LXXVII;

A = area of floor, in square feet;

D = depreciation factor. Suitable values are given in Table LXXVII;

E = efficiency of utilization depending on type of unit, shape of room, and character of interior finish. Suitable values are given in Table LXXVII;

L = lamp efficiency in lumens per watt. Values are given in Table LXXVIII.

TABLE LXXVII

Illumination Design Data

Room	Normal Intensity, Ft.-C.	Emergency Intensity, Ft.-C.	Depreciation Factor	Efficiency of Utilization, Per Cent
Turbine Room:				
Operation	4	2	1.3	40 to 50
Maintenance	8	2	1.3	40 to 50
Gate House	3	1	1.3	40 to 50
Wheel Pits	3	1	1.4	20 to 30
Passages, Tunnels	2	1	1.4	15 to 25
Oil Rooms	4	1	1.5	25 to 35
Rheostats, Reactors, Transformers	4	1	1.4	40 to 50
Oil Circuit-breakers	4	1	1.4	35 to 45
Bus Rooms	4	1	1.3	30 to 40
Battery Room	4	1	1.4	25 to 35
Switchboard Room:				
With ceiling lights	6	2	1.3	40 to 50
With glass ceiling	6	2	1.4	20 to 30
Offices	8	1	1.3	45 to 55

TABLE LXXVIII

Standard Lamp Data

(Manufacturer's Standard, Feb., 1926)

Bases are medium screw for sizes up to 200 watt and mogul screw for larger sizes. Standard lamps covered by this schedule are designed for burning in any position except that the 230-volt, 750-watt and 230-volt, 1000-watt lamps are designed for burning base up. These can be made for burning base down on request, in which case the light center distance is reduced by ½ in. Sizes 100 watt and smaller have inside frosted globes.

Volts	Watts	Initial Lumens, per Watt	Dimensions in Inches		
			Diameter	Length	Light Center
115	25	8.9	2⅜	3 15/16	2½
115	40	9.5	2⅝	4 7/16	2⅞
115	50	10.8	2⅝	4 15/16	3⅜
115	60	11.1	2⅝	5¼	3¾
115	100	13.2	2⅞	6 1/16	4⅜
115	150	15.2	3⅛	6 15/16	5¼
115	200	16.1	3¾	8⅛	6
115	300	17.4	4⅜	9 7/16	7
115	500	18.8	5	9 13/16	7
115	750	19.7	6½	13⅛	9½
115	1000	21.0	6½	13⅛	9½
230	100	10.6	3⅛	6 15/16	5¼
230	200	13.2	3¾	8⅛	6
230	300	14.5	4⅜	9 7/16	7
230	500	16.2	5	9 13/16	7
230	750	17.5	6½	13⅛	9½
230	1000	18.2	6½	13⅛	9½

CHAPTER XXXII

TRANSMISSION LINES

By Raymond A. Hopkins

425. Transmission Lines.—The hydro-electric generating station is usually located at a considerable distance from the distribution system into which it feeds and, therefore, the connecting transmission line is of great economic importance. Unlike the usual steam plant which radiates its energy over a number of comparatively small feeders, the hydro-electric plant usually delivers its entire output over two or three heavy transmission lines. The cost and performance of the line, therefore, have an important bearing on the cost of energy delivered at the load center.

426. Right-of-way.—The proper location of the right-of-way over which a transmission line is to be constructed is an economic problem of great importance. Careful reconnaissance work is needed to determine the best location. In flat and rolling country, aerial observation and photographic mapping are now used to advantage. In rough country, an aerial survey is of somewhat less value, although it may aid in the selection of the most direct route. Accessibility for delivery of materials and for patrolling is a controlling factor. The detailed survey following the reconnaissance should locate lakes, swamps, hills, towns, railroads, power and communication lines, and legal property lines. Profiles should be run for determining tower locations and heights to give the necessary clearance of the cables above the ground. The maps and profiles should show probable tower locations as selected by the field party.

The right-of-way for an important transmission line is sometimes owned by the power company, although in some cases the power company may preferably accept easements over certain parcels of privately owned property. The width of the right-of-way is determined by the ultimate number of circuits that may be run. It is desirable to place parallel tower lines so far apart that a falling tower of one line will not strike the other line. The edge of the right-of-way should be at a safe clearance distance, roughly 2 in. per thousand volts, beyond the extreme side-swing position of the center of the span. The right-of-way should be cleared of trees, and in some cases it is advisable to cut trees on adjacent property to prevent the possibility of their falling on the lines.

427. Frequency and Phase.—The frequency of the transmission line is generally determined by the frequency of the power system with which the line will be connected, as this consideration usually outweighs the various slight advantages of one frequency over another. (See Sec. 349.)

Three-phase transmission is practically standard for all important lines. Distribution lines are sometimes single-phase, two-phase, or direct-current, as governed by local conditions; but the tendency, for such lines also, is toward the uniform use of the three-phase system. The principal advantages of the three-phase system are high copper economy and small number of conductors. (See Sec. 350.)

428. Number and Arrangement of Circuits.—For maximum reliability it is desirable to divide the transmission system into at least two parallel circuits, each of such capacity that one may be retired for inspection and maintenance without curtailing the delivery of energy. Where a large block of power is transmitted a long distance, it becomes quite necessary to use more than one circuit in order to avoid excessive reactance drop.

A separate pole line or tower line is preferable for each circuit, in order to provide greater safety for the maintenance men and also to prevent trouble on one circuit from affecting the other. The latter consideration may, in extreme cases, justify separate routes. Where more than two circuits are used, double-circuit towers are often considered satisfactory and are less expensive than twice the number of single-circuit towers.

Clearances over highways, railroads, and other wires are specified in the rules of the National Safety Code. The lower the towers the more reliable will be the line.

The configuration of the wires is generally triangular, flat-vertical, or flat-horizontal. The triangular configuration is adapted to the lower and moderately high voltages, particularly where the conductors are supported by pin-type insulators. It may be symmetrical, that is, with equal spacings between conductors, or unsymmetrical. The flat-vertical configuration is adapted to two-circuit towers for high voltages. The middle conductor is usually offset from the plane of the other two by a distance of about one-third the spacing, to reduce the possibility of trouble from sleet and snow. The flat-horizontal configuration is adapted to single-circuit towers for high voltages. Transposition of the conductors of any configuration is often advisable in order to reduce disturbances on adjacent telephone lines. On very long lines it is sometimes advantageous to transpose at least twice, for the purpose of balancing the reactance and capacity equally among the three conductors and between the conductors and ground.

Spacing between conductors cannot be determined by any fixed rule. It should be roughly proportional to the voltage, but also depends upon the configuration, type of insulator, length of span, conditions of loading, and local regulations. A larger spacing is usually required horizontally than vertically. Larger spacing is also required for suspension insulators than for pin insulators.

Ground conductors are often used on steel tower lines as a means of protecting the power conductors from lightning, and also, in some cases, to increase the mechanical stability of the line. The ground conductor should be of high-strength steel and should be either galvanized or copper-covered. The connections at the towers should be firm and so arranged as to avoid wear from the swinging of the conductor. For further information on ground-

ing see Sec. 419, also "Present-day Practices in Grounding of Transmission Systems," *Trans.* A.I.E.E., Vol. XLII, April, 1923, p. 446.

429. Insulators.—The conductors must be attached to the supporting structures by means of insulators designed to withstand safely the mechanical stress and electric potential. Insulators are usually made of vitrified, glazed porcelain. Glass has also been used to a limited extent for high-voltage lines and quite generally for low-voltage distribution lines and for telephone lines. The two general types of insulators are pin-type and suspension-type.

The pin insulator is intended for mounting on a wood or steel pin and is provided with a groove at the top for attaching the conductor. As it is not practical, from the manufacturing standpoint, to make porcelain of greater thickness than about $\frac{3}{4}$ in., and as this thickness is not suitable for a voltage of more than 10,000 to 15,000, volts, the high-voltage insulator is made by assembling several concentric shells as shown in Fig. 458. The shells are cemented together with Portland cement, the surfaces in contact with the cement being sanded before firing, to give a strong bond. Objectives sought in the design of the pin insulator include maximum leakage distance with minimum leakage area, greater flash-over voltage than puncture voltage, avoidance of corona formation, maximum mechanical strength, and endurance of extreme and sudden temperature changes and weather conditions. Although the pin insulator is made for voltages as high as 88,000 volts, it is usually more economical to use the suspension type for voltages of over 60,000 volts.

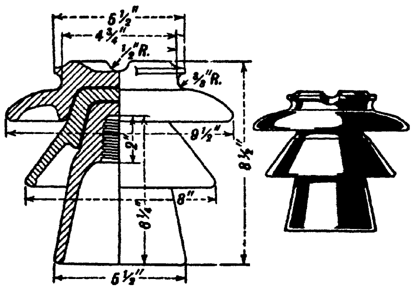


FIG. 458.—45,000-volt Pin Insulator.

Dry flash-over voltage (needle gap) . .	140,000
Wet flash-over voltage (needle gap) . .	95,000
Mechanical strength, lbs	3,000

(Locke Insulator Corp'n)

Insulator pins for voltages of 15,000 volts and less are generally made of wood, such as locust, oak, birch, maple, hickory, and eucalyptus. Wood pins are generally treated with paraffin, oil, or creosote. For higher voltages, a steel pin is generally required on account of the mechanical strength necessary. To secure a tight fit between the steel pin and the porcelain insulator without excessive pressure at spots, various devices are used, as, threading and splitting the pin top, providing a wire spring thread, providing a threaded wood or lead top, cementing into the insulator, or providing a separate thimble which is cemented into the insulator at the factory and threaded on to the pin in the field.

Suspension insulators are generally shaped like disks and are intended for connecting together in strings to suspend the conductor. A string may be used either vertically, as for an intermediate support, or nearly horizontally, as in a dead-end or strain support. Where great mechanical strength is required, a group may be formed by arranging several strings in parallel, care

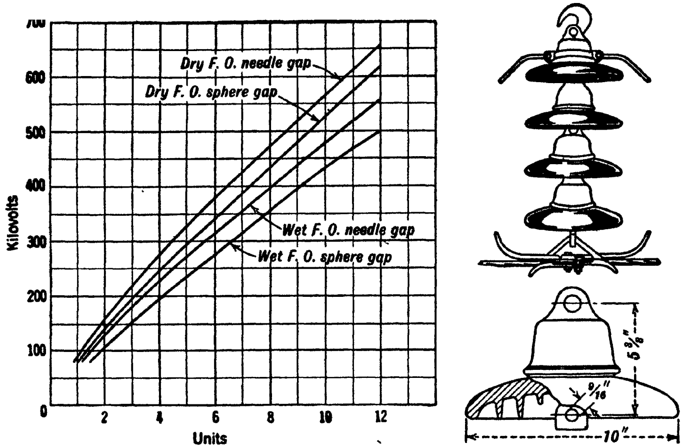


Fig. 459.—Cemented Type Suspension Insulator.

The curves show average dry and wet flash-over values with needle gap and with sphere gap.
Mechanical strength, lbs. 9,000
(Locke Insulator Corp'n).

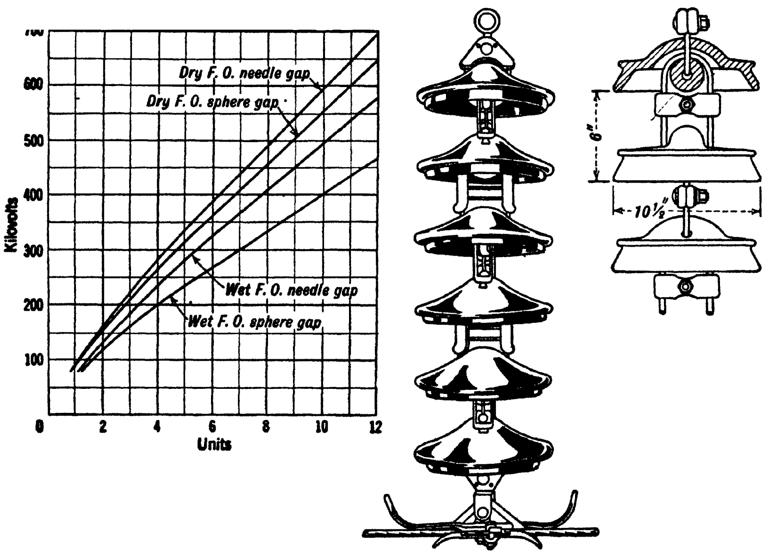


Fig. 460.—Cementless Type Suspension Insulator.

The curves show average dry and wet flash-over values with needle gap and with sphere gap. Mechanical strength, lbs. 10,000
(Locke Insulator Corp'n)

being taken to distribute the load equally among the various strings by means of equalizing bars at the ends.

Two general forms of disks are in common use. The cemented type is illustrated in Fig. 459, and the cementless type in Fig. 460. With the former type the cap and pin are cemented to the porcelain, while with the latter the porcelain is pierced to receive the hardware.

The flash-over voltage, as indicated by curves of Figs. 459 and 460, is roughly proportional to the number of disks or units comprising the string. The distribution of the voltage stress among the various units is, however, not at all uniform. The end units receive more potential than the middle units, and with long strings the conductor end receives more than the support end. This is caused by a distortion of the electrostatic field as influenced by the condenser action of the metal hardware between units. For very high potentials this phenomenon is of serious importance, as the unit next to the conductor may receive a potential exceeding its rating. A number of experiments have been made with various forms of electrostatic shields at each end of the string for grading the voltage, as discussed by R. J. C. Wood in *Trans.* A.I.E.E., Vol. XLI, p. 715, August, 1922. An example of insulator grading and its effect are shown in Fig. 461.

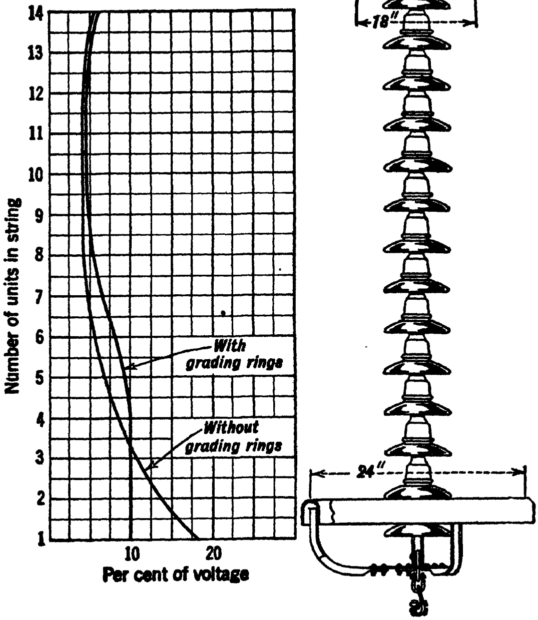


FIG. 461.—Distribution of Voltage on Insulator String with and without Grading Rings. (Locke Insulator Corp'n.)

Arcing horns are often used at each end of an insulator string to divert the arc of a flash-over away from the porcelain and the conductor, in order to avoid injury. Typical arcing horns are shown in Figs. 459 and 460.

Tests of insulators are of two classes: design tests made on a limited number of insulators to check the design; and routine tests made on all insulators to detect defects. The routine tests should include flash-over of each shell before assembly, flash-over of the completed insulator, and mechanical strength of the completed insulator. It is not recommended that any insulator be

tested to over 50 per cent of its ultimate mechanical strength before being put into service. A higher test is likely to cause permanent injury and, furthermore, is not needed to detect most defects. A selected few, however, should be tested to destruction. For important installations, it is advisable to apply a combined mechanical and electrical test to destruction, as both electrical failure and mechanical failure generally occur at lower values under these conditions. For standard conditions of test, reference should be made to the Standards of the A.I.E.E.

430. Corona.—Corona is the breaking down of the air as a dielectric near the surface of a conductor when the potential gradient exceeds a certain critical value. The phenomenon is accompanied by a bluish light and a hissing sound. It is often manifest in electrical machinery and in station wiring unless precautions are taken to guard against it. On high-voltage transmission lines it often becomes a factor in limiting the diameter of conductor or some other feature of design, as the power loss increases very rapidly with increase in voltage above the critical value. Very extensive experimentation by F. W. Peek, Jr., and others has resulted in the empirical formulae given below, which define both the critical voltage and the value of power loss.

The disruptive critical voltage is the voltage at which corona becomes apparent. During fog, rain, sleet, or snow, the critical voltage is from 10 to 20 per cent lower than in fair weather. The critical voltage and power loss for the conductors of a transmission line under conditions of fair weather are:

$$E_0 = 107\frac{17.9b}{459 + t}md \log_{10}\left(\frac{2D}{d}\right), \qquad (211)$$

$$P = 0.039\frac{459 + t}{17.9b}\sqrt{\frac{d}{2D}}(f + 25)(E - E_0)^2, \qquad (212)$$

where E_0 = disruptive critical voltage, in effective kilovolts between conductors;

b = barometric pressure in inches of mercury at 32° F. corresponding to the altitude (30.00 at sea level, 28.86 at 1000 ft., 27.77 at 2000 ft., 26.75 at 3000 ft., 25.76 at 4000 ft., 24.78 at 5000 ft., 23.85 at 6000 ft., 22.98 at 7000 ft., 22.12 at 8000 ft., 21.28 at 9000 ft., 20.50 at 10,000 ft.);

t = temperature of air, in degrees F.;

m = irregularity factor of conductor surface (1.0 for polished wire, 0.98 to 0.93 for weathered wire, 0.87 to 0.83 for stranded cable).

d = diameter of conductor in inches;

D = distance between centers of conductors in inches. For irregular spacing: $a, b, c, D = \sqrt[3]{abc}$;

P = power loss per mile of line, in kilowatts;

f = frequency, in cycles per second;

E = operating voltage, in effective kilovolts between conductors.

431. Voltage and Conductor Size.—The choice of voltage and conductor size, while usually modified by such local considerations as voltage regulation and connecting lines, nevertheless rests fundamentally upon pure economics as expressed by Kelvin's law. As the voltage increases, the power loss decreases, while the cost of terminal equipment increases. As the conductor size increases, the power loss decreases, but the cost of conductor increases. There is, therefore, an economical voltage and conductor size for which the total annual cost is a minimum.

Professors F. K. Kirsten and E. A. Loew, in University of Washington Experimental Station Bulletin No. 32, have presented a simple method of arriving simultaneously at the economical voltage and conductor size by making use of the relation between voltage and conductor size as given in Mr. Peek's Law of Corona, on the assumption that a properly designed line will operate at just a safe margin below the corona voltage. Through the courtesy of the authors, a brief outline of this method is presented below, with some slight modifications consisting principally of expressing the conductor size in terms of area instead of diameter. The method is also extended to take account of various kinds of conductors, such as solid, stranded, steel-reinforced, rope-core and hollow; various conductor spacings; and various substation layouts.

The notations and equations, with explanatory notes, are as follows:

Notation

P = root-mean-square value of power delivered by the line, in kilowatts. This must be determined from an exhaustive study of river flow and load-demand conditions. It is the r.m.s. ordinate of the average daily load curve.

p = average power factor at the load end, expressed as a decimal.

E = voltage between wires at load end, in kilovolts.

I = current of entire circuit, in amperes $= \dfrac{P}{pE}$.

E_0 = disruptive critical corona voltage, in kilovolts between wires.

l = length of line, in miles.

a = cross-sectional area of conducting material of conductor, in circular mils.

S = spiraling factor of conductor, as follows:

Solid conductor	1.00
Stranded conductor	1.02

s = stranding factor or ratio of over-all diameter of conductor to diameter of equivalent solid conductor as follows:

Solid conductor	1.00
Standard stranded conductor	*1.15
Steel-reinforced stranded conductor	*1.23
Rope-core stranded conductor	*1.35
Hollow stranded conductor	*1.35

* These values depend on the design of the cable, but are sufficiently accurate for this purpose.

ρ = resistivity of conductor material, in ohms per mil-foot as follows:

	25° C.	65° C.
Hard-drawn copper, 97.3 per cent...	10.87	12.54
Hard-drawn aluminum, 61 per cent..	17.33	19.99

e = energy cost, in dollars per kilowatt-hour at receiving end of line.

c = conductor cost, in dollars per mil-foot of conducting material. The following figures are fair estimates:

Solid copper........................	45.4×10^{-8}
Stranded copper.....................	52.2×10^{-8}
Stranded aluminum...................	28.9×10^{-8}
Steel-reinforced aluminum...........	30.1×10^{-8}
Rope-core copper....................	55.4×10^{-8}
Hollow copper.......................	59.0×10^{-8}

r = rate of interest and other fixed charges, expressed as a decimal.

K = constant determined from the relation between line voltage and cost of terminal equipment, as expressed by the equation:

$$\text{Cost} = K_1 + KE^2.$$

The variable part of the cost, namely KE^2, is the only part to be considered. Careful estimates of typical substation layouts including transformers, breakers, disconnecting switches, arresters, and structures at both ends of the line, show that K varies from 8 for a simple layout to 13 for a complex layout.

$U = \dfrac{E}{\sqrt{a}}$ = constant obtained from curves of Fig. 462 and represents the relation between voltage and conductor size to allow a safe margin below the corona voltage. The curves are obtained from the equation:

$$U = \frac{E}{\sqrt{a}} = 0.00258sb \log_{10}\left(\frac{2000D}{s\sqrt{a}}\right),$$

which is derived from the corona equation (211) by the following substitutions:

$$E = 0.85E_0;$$
$$t = 77° \text{ F.};$$
$$m = 0.85;$$
$$d = s\sqrt{a};$$

To use Fig. 462, select the average altitude of the line, move upward on the figure to intersection with curve representing desired relation between D and E, thence horizontally to intersection with curve s corresponding to type of conductor, thence upward to scale of U.

F, G, H = constants as defined by the three cost equations given below:

L_1 = annual cost, in dollars, of energy wasted in resistance of conductors.

L_2 = annual fixed charge, in dollars, on conductors, chargeable to their size.

L_3 = annual fixed charge, in dollars, on terminal equipment, chargeable to line voltage.

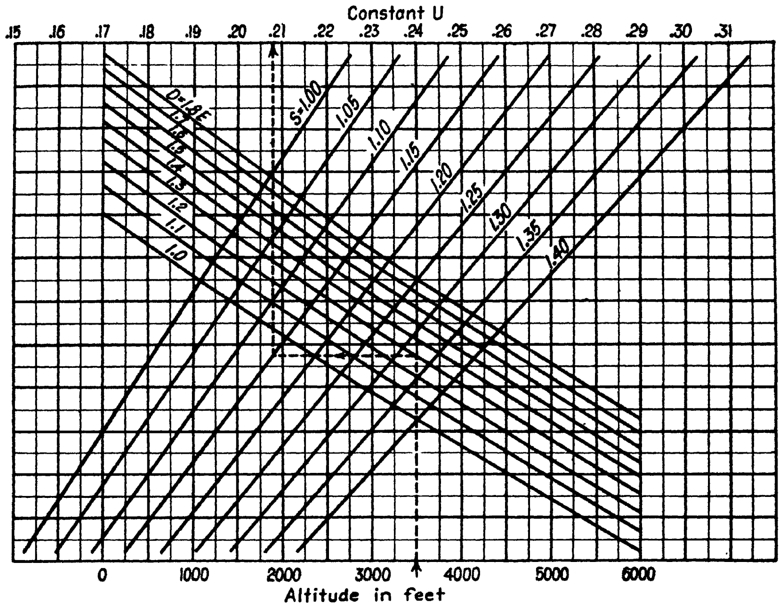


FIG. 462.

The Three Cost Equations

L_1 = resistance × current² × hours × energy cost

$$= 5280\rho S\frac{l}{a} \times I^2 \times 8760 \times e10^{-3}$$

$$= (46253\rho Sle)\frac{I^2}{a}$$

$$= F\frac{I^2}{a}.$$

L_2 = quantity × conductor cost × rate

$$= 5280la \times 3c \times r$$

$$= (15840lcr)a$$

$$= Ga.$$

L_3 = constant × voltage² × rate

$$= K \times E^2 \times r$$

$$= (KU^2r)a$$

$$= Ha.$$

Statement of Kelvin's Law

$$L_1 = L_2 + L_3;$$

$$F\frac{I^2}{a} = Ga + Ha;$$

$$a^2 = I^2 \times \frac{F}{G+H};$$

$$a^3 = \frac{E^2I^2}{U^2} \times \frac{F}{G+H};$$

$$a = \sqrt[3]{\frac{P^2}{p^2U^2} \times \frac{F}{G+H}} = \text{economical conductor size.} \quad . \quad . \quad . \qquad (213)$$

$$E = U\sqrt{a} = \text{economical voltage.} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (214)$$

432. Fundamental and Derived Constants.—The fundamental constants of a transmission line are its resistance, reactance, impedance, conductance, susceptance, and admittance. The constants are independent of voltage, current, and power factor, and are determined largely by the material, size, and spacing of the conductors. Many tables are available, giving the values of these constants, of which those contained in "Electrical Characteristics of Transmission Circuits," by William Nesbit, are especially convenient. The following paragraphs further define the constants and give equations from which they may be computed if desired.

Resistance, R, is expressed in ohms of one conductor and is determined from the equation:

$$R = 5.28\rho S\left(\frac{l}{a}\right)10^3, \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (215)$$

where ρ = resistivity of the conducting material of the conductor in ohms per mil-foot, as follows:

	25° C.	65° C.
Hard-drawn copper 97.3 per cent conductivity	10.87	12.54
Hard-drawn aluminum 61 per cent conductivity	17.33	19.99

S = spiraling factor as follows:

Solid conductor	1.00
Stranded conductor	1.02

l = length of line, in miles;

a = cross-sectional area of conducting material of conductor, in circular mils.

The resistance voltage is in phase with the current and is equal to the product of the resistance and the current.

Reactance, X, is expressed in ohms of one conductor and is determined from the equation:

$$X = 2\pi fl\left[8.05 + 74.1 \log_{10}\left(\frac{2D_8}{d}\right)\right]10^{-5}, \quad . \quad . \quad . \quad . \qquad (216)$$

where $\pi = 3.1416$;
f = frequency, in cycles per second;
l = length of line, in miles;
D = spacing of conductors, in inches. For irregular spacing: a, b, c, $D = \sqrt[3]{abc}$;
s = stranding factor or ratio of over-all diameter of cable to diameter of equivalent solid conductor, as follows:

Solid conductor	1.00
Standard stranded conductor	*1.15
Steel-reinforced stranded conductor	*1.23
Rope-core stranded conductor	*1.35
Hollow stranded conductor	*1.35

* These values are approximate. Exact values depend upon the design of the cable.

d = over-all diameter of conductor, in inches.

The reactance voltage is in leading quadrature with the current and is equal to the product of the reactance and the current.

Impedance, Z, is expressed in ohms of one conductor and is determined from the equations:

$$\left.\begin{aligned} \underset{\cdot}{Z} &= R + jX \\ Z &= \sqrt{R^2 + X^2} \end{aligned}\right\} \quad \ldots\ldots \quad (217)$$

The impedance voltage leads the current and is equal to the product of the impedance and the current.

Conductance, G, is expressed in mhos of one conductor and is determined from the leakage of current from conductor to conductor, either through the air or over the surfaces of insulators. Where the conductors are properly separated and insulated the conductance is so small at the commercial frequencies and lengths of power lines that it is generally considered that:

$$G = 0. \quad \ldots\ldots \quad (218)$$

The conductance current, if it were appreciable, would be in phase with the voltage and equal to the product of the conductance and the voltage.

Susceptance, B, is expressed in mhos of one conductor and is determined from the equation:

$$B = 2\pi f l \left[\frac{3.88}{\log_{10}\left(\frac{2D}{d}\right)} \right] 10^{-8}, \quad \ldots\ldots \quad (219)$$

where $\pi = 3.1416$;
f = frequency, in cycles per second;
l = length of line, in miles;
D = spacing of conductors in inches. For irregular spacing: a, b, c, $D = \sqrt[3]{abc}$;
d = over-all diameter of conductor, in inches.

The susceptance current, commonly called the charging current, is in

leading quadrature with the voltage and is equal to the product of the susceptance and the voltage.

Admittance, Y, is expressed in mhos of one conductor and is determined from the equation:

$$\left.\begin{aligned} \dot{Y} &= G + jB \\ Y &= \sqrt{G^2 + B^2} \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (220)$$

The admittance current leads the voltage and is equal to the product of the admittance and the voltage.

The derived constants, A, B and C, are obtained from the fundamental constants by means of the following equations:

$$\left.\begin{aligned} \dot{A} &= a_1 + ja_2 = \left(1 + \frac{\dot{Y}\dot{Z}}{2} + \frac{\dot{Y}^2\dot{Z}^2}{24} + \frac{\dot{Y}^3\dot{Z}^3}{720} + \frac{\dot{Y}^4\dot{Z}^4}{40320} + \ldots\right) \\ \dot{B} &= b_1 + jb_2 = \dot{Z}\left(1 + \frac{\dot{Y}\dot{Z}}{6} + \frac{\dot{Y}^2\dot{Z}^2}{120} + \frac{\dot{Y}^3\dot{Z}^3}{5040} + \frac{\dot{Y}^4\dot{Z}^4}{362880} + \ldots\right) \\ \dot{C} &= c_1 + jc_2 = \dot{Y}\left(1 + \frac{\dot{Y}\dot{Z}}{6} + \frac{\dot{Y}^2\dot{Z}^2}{120} + \frac{\dot{Y}^3\dot{Z}^3}{5040} + \frac{\dot{Y}^4\dot{Z}^4}{362880} + \ldots\right) \end{aligned}\right\} \quad (221)$$

These constants take into account the fact that the fundamental constants are actually distributed along the entire length of the line. They are used in the exact performance equations given in later paragraphs. Any degree of accuracy can be obtained by using a sufficient number of terms of the series. It is rarely necessary, however, to use more than three terms.

In the solution of a single-phase or a polyphase circuit, it is assumed that the circuit is symmetrical about its neutral and, therefore, the solution of one leg of the circuit determines the performance of the entire circuit. Therefore, the constants R, X, G and B are always given for one wire only and are used as one-wire values in all equations. The kv-a., voltage, and current may be expressed in terms of one leg also, and the result will then be in terms of one leg. If preferred, however, the one-wire constants can be used with kv.-a., voltage and current expressed in terms of the entire circuit, and the results will then be in terms of the entire circuit. This latter method is preferred by many engineers. The two methods are fully defined in the following tabulation:

Method 1.—*In Terms of One Leg*

K = volt-amperes per conductor
= volt-amperes of entire circuit divided by:
2 for a 2-wire, single-phase circuit
3 for a 3-wire, three-phase circuit
4 for a 4-wire, two-phase circuit.

E = volts from conductor to neutral
= volts between conductors divided by:
2 for a 2-wire, single-phase circuit
$\sqrt{3}$ for a 3-wire, three-phase circuit
2 for a 4-wire, two-phase circuit.

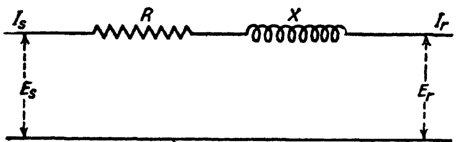
I = amperes per conductor;
$K = EI.$

Method 2.—In Terms of Entire Circuit

K = volt-amperes of the entire circuit;
E = volts between conductors;
I = amperes of the entire circuit
= amperes per conductor multiplied by:
1 for a 2-wire, single-phase circuit
$\sqrt{3}$ for a 3-wire, three-phase circuit
2 for a 4-wire, two-phase circuit.

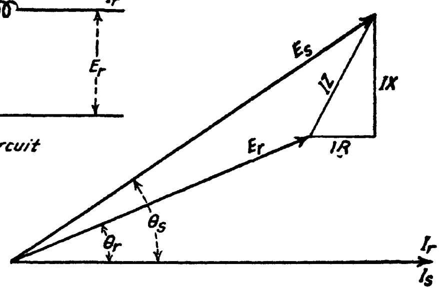
$K = EI.$

Conditions of power, voltage, and current are usually fixed at the receiving end by requirements of the load, and in this case computation is made to



Short line - equivalent circuit

FIG. 463.



Short line - vector diagram

FIG. 464.

determine corresponding conditions at the sending end. Occasionally, however, conditions are fixed at the generating end and computations must be made to obtain the corresponding conditions at the receiving end.

433. Approximate Performance Equations.—The performance of a short line, particularly when operated at low frequency, may be determined by approximate equations which neglect the distributed nature of the constants and ignore the susceptance. The derived constants are not needed for this solution. The error is in the order of:

0.1 per cent for a 1200 cycle-mile line;
0.5 per cent for a 3100 cycle-mile line;
1.0 per cent for a 4200 cycle-mile line.

Thus, with an error not exceeding 0.5 per cent, the approximate solution may be used for a 60-cycle, 52-mile line or for a 25-cycle, 124-mile line.

The equivalent circuit of a short line may be represented as in Fig. 463. The current and voltage vector diagram for the case of lagging power factor at both ends of the line is shown in Fig. 464. The figures represent one leg of the

circuit, the imaginary neutral having zero resistance and reactance. The notation and equations are as follows:

Notation

R = resistance, X = reactance — Fundamental constants as defined in Sec. 432;

P = power of entire circuit in watts;

Q = reactive volt-amperes of entire circuit. (Positive for leading power factor and negative for lagging power factor);

$\dot{K} = P + jQ$ = volt-amperes of entire circuit;

PF = power factor;

$\theta = \cos^{-1} PF$; $\sin\theta$ is positive when PF is lagging and negative when PF is leading;

E = voltage between conductors, in volts;

I = current of entire circuit, in amperes;

r and s used as subscripts refer to receiving end and sending end respectively.

Equations for Fixed Conditions at Receiving End

$$\left.\begin{aligned}
P_r &= K_r \cos\theta_r = E_r I \cos\theta_r \\
E_s &= \sqrt{(E_r\cos\theta_r + IR)^2 + (E_r \sin\theta_r + IX)^2} \\
I_s &= I_r = I \\
PF_s &= \cos\theta_s = \frac{E_r\cos\theta_r + IR}{E_s} \\
P_s &= E_s I\cos\theta_s = E_r I \cos\theta_r + I^2R
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (222)$$

Equations for Fixed Conditions at Sending End

$$\left.\begin{aligned}
P_s &= K_s \cos\theta_s = E_s I \cos\theta_s \\
E_r &= \sqrt{(E_s\cos\theta_s - IR)^2 + (E_s \sin\theta_s - IX)^2} \\
I_r &= I_s = I \\
PF_r &= \cos\theta_r = \frac{E_s\cos\theta_s - IR}{E_r} \\
P_r &= E_r I\cos\theta_r = E_s I \cos\theta_s - I^2R
\end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (223)$$

The use of the above equations is illustrated by the following typical short-line problem:

A transmission line 40 miles long consists of three conductors of No. 1/0 B. & S., 106,000 cir. mils, hard-drawn stranded copper, over-all diameter 0.373 in., arranged at the vertices of a triangle with sides 80, 85 and 90 in. respectively. Determine the line performance with a frequency of 60 cycles per second and with fixed conditions of 15,000 kw., 80 per cent lagging power factor, 66,000 volts (*a*) at the receiving end, and (*b*) at the sending end.

The line constants are computed from Eqs. (215) and (216) as follows:

$$R = 5.28 \times 10.87 \times 1.02\left(\frac{40}{106{,}000}\right)10^3$$
$$= 22.1;$$
$$X = 2 \times 3.1416 \times 60 \times 40\left[8.05 + 74.1 \log_{10}\left(\frac{2 \times 84.92 \times 1.15}{0.373}\right)\right]10^{-5}$$
$$= 31.6.$$

(*a*) Performance with fixed conditions at the receiving end is computed from Eq. (222) as follows:

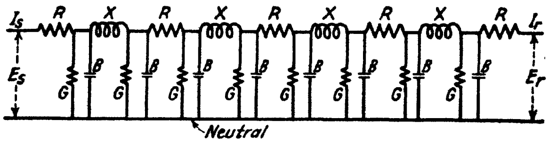
$$P_r = 15{,}000{,}000;$$
$$\cos\theta_r = 0.8;\ \sin\theta = 0.6;$$
$$E_r = 66000;$$
$$I = \frac{15{,}000{,}000}{0.8 \times 66{,}000} = 284;$$
$$E_s = \sqrt{(66{,}000 \times 0.8 + 284 \times 22.1)^2 + (66{,}000 \times 0.6 + 284 \times 31.6)^2}$$
$$= 76{,}482,$$
$$\cos\theta_s = \frac{(66{,}000 \times 0.8) + (284 \times 22.1)}{76{,}482}$$
$$= 0.772,$$
$$P_s = 76{,}482 \times 284 \times 0.772$$
$$= 16{,}768{,}500.$$

(*b*) Performance with fixed conditions at the sending end is computed from Eq. (223) as follows:

$$P_s = 15{,}000{,}000;$$
$$\cos\theta_s = 0.8;\ \sin\theta_s = 0.6;$$
$$E_s = 66{,}000;$$
$$I = \frac{15{,}000{,}000}{0.8 \times 66{,}000} = 284;$$
$$E_r = \sqrt{(66{,}000 \times 0.8 - 284 \times 22.1)^2 + (66{,}000 \times 0.6 - 284 \times 31.6)}$$
$$= 55{,}699;$$
$$\cos\theta_r = \frac{(66{,}000 \times 0.8) - (284 \times 22.1)}{55{,}699}$$
$$= 0.835;$$
$$P_r = 55699 \times 284 \times 0.835$$
$$= 13{,}208{,}500.$$

The calculation should be checked graphically by a simple diagram, as shown in Fig. 464. It should be remembered that for fixed conditions at the sending end the vectors IR and IX must be drawn backward, that is, 180 degrees from the positions shown in the figure, as their signs are negative in Eq. 223.

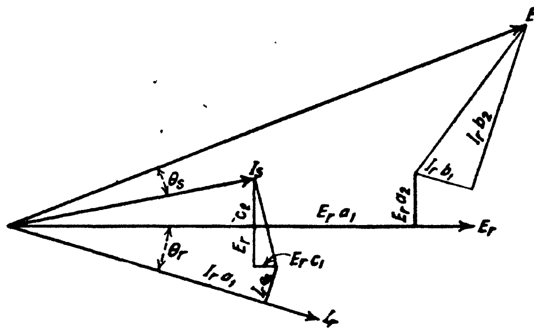
434. Exact Performance Equations.—The performance of long transmission lines, particularly when operated at 60 cycles per second, can be accurately determined only by the use of exact equations which take into account the distributed nature of the constants, including susceptance. The derived constants are required for this solution. The limits of length and frequency beyond which the exact methods are needed to avoid excessive error are given in Sec. 433.



Long line – equivalent circuit

FIG. 465.

The equivalent circuit of a long line may be represented as in Fig. 465. The voltage and current vector diagram for the case of lagging power factor at both ends of the line is shown in Fig. 466. The notation and equations are as follows:



Long line – vector diagram

FIG. 466.

Notation

R = resistance, X = reactance, G = conductance, B = susceptance: Fundamental constants as defined in Sec. 432.

$\dot{A} = a_1 + ja_2$, $\dot{B} = b_1 + jb_2$, $\dot{C} = c_1 + jc_2$: Derived constants as defined in Sec. 432.

P = power of entire circuit, in watts;

Q = reactive volt-amperes. (Positive for leading power factor and negative for lagging power factor.)

$\dot{K} = P + jQ$ = volt-amperes of entire circuit;

PF = power factor;

$\theta = \cos^{-1} PF$;

e_1 = in-phase component of voltage;

e_2 = quadrature component of voltage;

$\dot{E} = e_1 + je_2$ = voltage between conductors in volts. (As E_r is chosen as reference vector, $e_{r1} = E_r$ and $e_{r2} = 0$. e_{s2} is positive when E_s leads E_r and negative when E_s lags E_r.);

i_1 = in-phase component of current;

i_2 = quadrature component of current;

$\dot{I} = i_1 + ji_2$ = current of entire circuit in amperes. (i_{r2} is positive when I_r leads E_r and negative when I_r lags E_r. i_{s2} is positive when I_s leads E_r and negative when I_s lags E_r);

r and s, used as subscripts, refer to receiving end and sending end, respectively.

Equations for Fixed Conditions at Receiving End

$$\left.\begin{aligned}
P_r &= K_r \cos\theta_r\ E_rI_r \cos\theta_r \\
\dot{E}_s &= \dot{E}_r\dot{A} + \dot{I}_r\dot{B} \\
\dot{I}_s &= \dot{I}_r\dot{A} + \dot{E}_r\dot{C} \\
PF_s &= \cos\theta_s = \frac{e_{s1}i_{s1} + e_{s2}i_{s2}}{E_sI_s} \\
P_s &= E_sI_s \cos\theta_s = e_{s1}i_{s1} + e_{s2}i_{s2}
\end{aligned}\right\} \quad (224)$$

Equations for Fixed Conditions at Sending End

$$\left.\begin{aligned}
P_s &= K_s \cos\theta_s = E_sI_s \cos\theta_s \\
\dot{E}_r &= \dot{E}_s\dot{A} - \dot{I}_s\dot{B} \\
\dot{I}_r &= \dot{I}_s\dot{A} - \dot{E}_s\dot{C} \\
PF_r &= \cos\theta_r = \frac{e_{r1}i_{r1} + e_{r2}i_{r2}}{E_rI_r} \\
P_r &= E_rI_r \cos\theta_r = e_{r1}i_{r1} + e_{r2}i_{r2}
\end{aligned}\right\} \quad (225)$$

The use of the above equations is illustrated by the following typical long-transmission-line problem:

A transmission line 200 miles long consists of three conductors of 605,000 cir. mils aluminum cable, steel-reinforced, over-all diameter 0.953 in., arranged in a plane with spacing 14, 14 and 28 ft. respectively. Determine the line performance with a frequency of 60 cycles per second, and with fixed conditions of 40,000 kw., 90 per cent lagging power factor, 132,000 volts (*a*) at the receiving end and (*b*) at the sending end.

The line constants are computed from Eqs. (215), (216), (217), (218), (219), (220) and (221) as follows:

$$R = 5.28 \times 17.33 \times 1.02\left(\frac{200}{605{,}000}\right)10^3$$

$$= 30.854;$$

$$X = 2\times 3.1416\times 60\times 200\left[8.05 + 74.1 \log_{10}\left(\frac{2 \times 211.68 \times 1.23}{0.953}\right)\right]10^{-6}$$

$$= 159{,}015;$$

$$\dot{Z} = 30.854 + j159.015;$$

$$G = 0;$$

$$B = 2 \times 3.1416 \times 60 \times 200 \left[\frac{3.88}{\log_{10}\left(\frac{2 \times 211.68}{0.953}\right)} \right] 10^{-8}$$

$$= 0.0011049;$$

$$\dot{Y} = 0 + j0.0011049;$$

$$\dot{A} = 0.91338 + j0.01655;$$

$$A = 0.91354;$$

$$\dot{B} = 29.07041 + j154.5701;$$

$$B = 157.28;$$

$$\dot{C} = -0.000006 + j0.001073;$$

$$C = 0.001073.$$

(*a*) Performance with fixed conditions at the receiving end is computed from Eq. (224) as follows:

$$P_r = 40{,}000{,}000;$$

$$\cos\theta_r = 0.9;\ \sin\theta_r = 0.436;$$

$$E_r = 132{,}000;$$

$$e_{r1} = 132{,}000;$$

$$e_{r2} = 0;$$

$$\dot{E}_r = 132{,}000;$$

$$I_r = \frac{40{,}000{,}000}{0.9 \times 132{,}000} = 336.70;$$

$$i_{r1} = 0.9 \times 336.7 = 303.03;$$

$$i_{r2} = 0.436 \times 336.7 = 146.80;$$

$$\dot{I}_r = 303.03 - j146.80;$$

$$\dot{E}_s = (132{,}000)(0.91338 + j0.01655) + (303.03 - j146.80)(29.07041 + j154.5701)$$

$$= 152{,}066 + j44{,}756;$$

$$E_s = 158{,}516;$$

$$\dot{I}_s = (303.03 - j146.80)(0.91338 + j0.01655) + (132{,}000)(-0.000006 + j0.001073)$$

$$= 278.4 + j12.57;$$

$$I_s = 278.7;$$

$$\cos\theta_s = \frac{(152{,}066 \times 278.4) + (44{,}756 \times 12.57)}{158{,}516 \times 278.7}$$

$$= 0.971 \text{ lagging};$$

$$P_s = (152{,}066 \times 278.4) + (44{,}756 \times 12.57)$$

$$= 42{,}898{,}000.$$

(*b*) Performance with fixed conditions at the sending end is computed from Eq. (225) as follows:

$$P_s = 40{,}000{,}000;$$

$$\cos\theta_s = 0.9;\ \sin\theta_s = 0.436;$$

$$E_s = 132{,}000;$$
$$e_{s1} = 132{,}000;$$
$$e_{s2} = 0;$$
$$\dot{E}_s = 132{,}000;$$
$$I_s = \frac{40{,}000{,}000}{0.9 \times 132{,}000} = 336.70;$$
$$i_{s1} = 0.9 \times 336.70 = 303.03;$$
$$i_{s2} = 0.436 \times 336.70 = 146.80\cdot$$
$$\dot{I}_s = 303.03 - j146.80;$$
$$\dot{E}_r = (132{,}000)(0.91338 + j0.01655) - (303.03 - j146.80)(29.07041 + j154.5701);$$
$$= 89{,}066 - j40{,}387;$$
$$E_r = 97{,}795;$$
$$\dot{I}_r = (303.03 - j146.80)(0.91338 + j0.01655) - (132{,}000)(-0.000006 + j0.001073);$$
$$= 280.0 - j270.7;$$
$$I_r = 389.4;$$
$$\cos\theta_r = \frac{(89{,}066 \times 280) + (40{,}387 \times 270.7)}{97{,}795 \times 389.4};$$
$$= 0.942 \text{ lagging};$$
$$P_s = (89{,}066 \times 278.4) + (40{,}387 \times 270.7)$$
$$= 35{,}871{,}000.$$

The calculation should be checked graphically by a simple diagram as shown in Fig. 466. It should be remembered that for fixed conditions at the sending end the vectors $\dot{I}\dot{B}$ and $\dot{E}\dot{C}$ must be drawn backward, that is, 180 degrees from the positions shown in the figure, as their signs are negative in Eq. (225).

435. Transformer Constants.—The transformers at each end of the transmission line must be considered in determining the performance of the system as a whole. The constants of a bank of transformers may be expressed in the same terms as the fundamental constants of the line, as follows:

$$\left.\begin{aligned} R &= \frac{rE^2}{100K} \\ X &= \frac{xE^2}{100K} \\ \dot{Z} &= R + jX \\ G &= \frac{gK}{100E^2} \\ B &= \frac{bK}{100E^2} \\ \dot{Y} &= G - jB \end{aligned}\right\} \quad \cdots\cdots\cdots \quad (226)$$

where R = resistance per conductor in ohms;
X = reactance per conductor, in ohms;
G = conductance per conductor, in mhos;
B = susceptance per conductor, in mhos;
r = resistance volts, in per cent of rated volts;
x = reactance volts, in per cent of rated volts;
g = iron-loss current, in per cent of rated current;
b = magnetizing current, in per cent of rated current;
E = rated voltage between conductors;
K = rated capacity, in volt-amperes.

Approximate values of r, x, g, and b suitable for preliminary transmission-system calculations are given in Table LXXIX. As these characteristics vary considerably with the transformer design, actual values for final calculations should be obtained from the transformer manufacturer.

TABLE LXXIX

TYPICAL TRANSFORMER CHARACTERISTICS

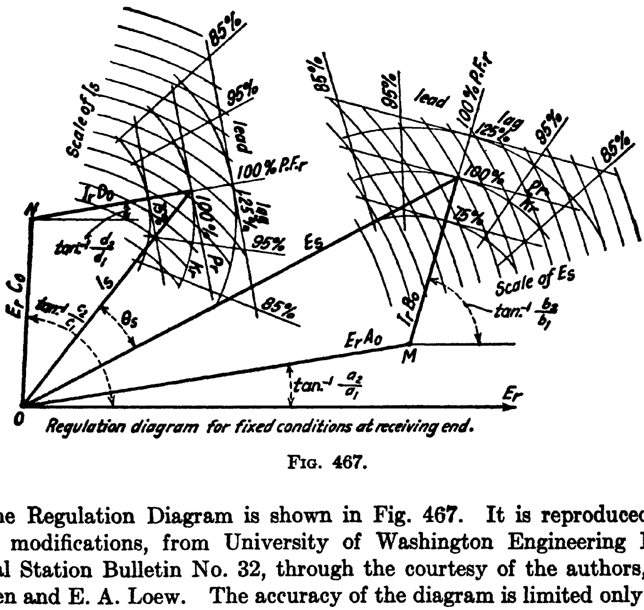
60-Cycle, Single-phase

Rated Kv-a.	Per Cent Resistance	Per Cent Reactance	Per Cent Iron Loss	Per Cent Magnetizing Current
W. C. 110 Kv.				
500	1.5	8.0	1.0	8.0
1,000	1.4	9.0	0.8	7.3
2,000	1.2	9.0	0.7	6.2
5,000	1.0	9.5	0.57	5.0
10,000	0.85	9.5	0.48	4.3
20,000	0.75	10.0	0.42	3.6
30,000	0.70	10.0	0.38	3.3
W. C. 220 Kv				
5,000	1.2	10.5	0.81	6.9
10,000	1.0	11.0	0.67	5.9
20,000	0.86	12.5	0.52	4.7
30,000	0.80	13.0	0.44	4.2
S. C. 110 Kv.				
1,000	0.82	10.0	0.92	9.0
2,000	0.75	10.0	0.67	5.0
5,000	0.63	10.0	0.45	3.0
10,000	0.52	10.0	0.36	2.4
20,000	0.42	10.0	0.31	2.1
30,000	0.39	10.0	0.30	2.0

436. Networks.—A convenient method of determining the performance of a network consisting of transmission lines, transformers, and other impedances and admittances has been developed by Evans and Sels in *The Electric Journal*, for August, 1921, page 356. The method consists of combining the constants of the various elements of the system so as to give four general circuit con-

stants, A_0, B_0, C_0, and D_0, which may be used in the usual exact transmission-line equations. The more important elements with their equations are given in Table LXXX. For a system symmetrical about its center, such as a simple transmission line, the constant D_0 is equal to A_0.

437. Regulation and Power Diagrams.—Graphical methods of determining the performance of transmission systems are exceedingly useful because of their simplicity and the variety of solutions that may be obtained from a single diagram. It is always desirable to check at least one point on the diagram by the analytical method, to prove its accuracy. Many diagrams and charts have been devised, each having particular merit for certain kinds of analyses. The Regulation and Power Diagrams presented in the following paragraphs are particularly useful for the usual transmission problems.



Regulation diagram for fixed conditions at receiving end.

Fig. 467.

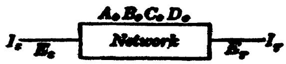
The Regulation Diagram is shown in Fig. 467. It is reproduced, with slight modifications, from University of Washington Engineering Experimental Station Bulletin No. 32, through the courtesy of the authors, F. K. Kirsten and E. A. Loew. The accuracy of the diagram is limited only by the degree of care exercised in constructing and reading it, as it is based on the exact performance equations. The diagram may be constructed to show the regulation of any form of network by first computing the four general circuit constants, A_0, B_0, C_0, and D_0, as explained in Sec. 436. The subsequent operations in constructing and reading the diagram for fixed conditions at the receiving end are as follows:

1. Using a sheet of plain paper, lay off E_r horizontally from O to a convenient voltage scale.
2. Construct angle $\tan^{-1}\dfrac{(a_2)}{(a_1)}$.
3. Lay off E_rA_0 to the voltage scale.

TABLE LXXX

Circuit Constants for Elementary Networks

General Circuit Equations:

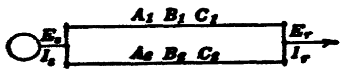


$$E_s = E_r A_0 + I_r B_0$$
$$I_s = I_r D_0 + E_r C_0$$
$$E_r = E_s D_0 - I_s B_0$$
$$I_r = I_s A_0 - E_s C_0$$

Two Transmission Lines in Series:

$$A_0 = a_1 + ja_2 = A_1 A_2 + C_1 B_2$$
$$B_0 = b_1 + jb_2 = B_1 A_2 + D_1 B_2$$
$$C_0 = c_1 + jc_2 = A_1 C_2 + C_1 D_2$$
$$D_0 = d_1 + jd_2 = B_1 C_2 + D_1 D_2$$

Two Transmission Lines in Multiple:



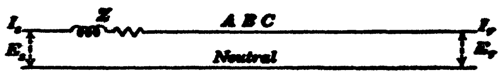
$$A_0 = a_1 + ja_2 = \frac{A_1 B_2 + B_1 A_2}{B_1 + B_2}$$

$$B_0 = b_1 + jb_2 = \frac{B_1 B_2}{B_1 + B_2}$$

$$C_0 = c_1 + jc_2 = C_1 + C_2 + \frac{(A_1 - A_2)(D_2 - D_1)}{B_1 + B_2}$$

$$D_0 = d_1 + jd_2 = \frac{B_1 D_2 + D_1 B_2}{B_1 + B_2}$$

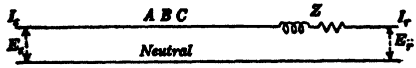
Transmission Line with Series Impedance at Sending End:



$$A_0 = a_1 + ja_2 = A + CZ$$
$$B_0 = b_1 + jb_2 = B + AZ$$
$$C_0 = c_1 + jc_2 = C$$
$$D_0 = d_1 + jd_2 = A$$

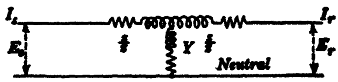
TABLE LXXX—*Continued*

Transmission Line with Series Impedance at Receiving End:



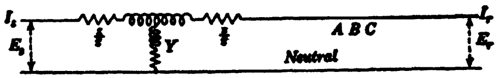
$$A_0 = a_1 + ja_2 = A$$
$$B_0 = b_1 + jb_2 = B + AZ$$
$$C_0 = c_1 + jc_2 = C$$
$$D_0 = d_1 + jd_2 = A + CZ$$

One Bank of Transformers:



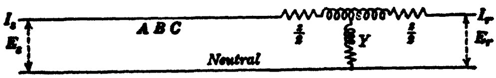
$$A_0 = a_1 + ja_2 = 1 + \frac{ZY}{2}$$
$$B_0 = b_1 + jb_2 = Z\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = Y$$
$$D_0 = d_1 + jd_2 = 1 + \frac{ZY}{2}$$

Transmission Line with Transformers at Sending End



$$A_0 = a_1 + ja_2 = A\left(1 + \frac{ZY}{2}\right) + CZ\left(1 + \frac{ZY}{4}\right)$$
$$B_c = b_1 + jb_2 = B\left(1 + \frac{ZY}{2}\right) + AZ\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = C\left(1 + \frac{ZY}{2}\right) + AY$$
$$D_0 = d_1 + jd_2 = A\left(1 + \frac{ZY}{2}\right) + BY$$

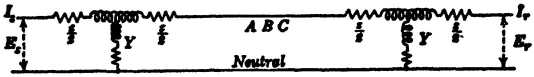
Transmission Line with Transformers at Receiving End:



$$A_0 = a_1 + ja_2 = A\left(1 + \frac{ZY}{2}\right) + BY$$
$$B_0 = b_1 + jb_2 = B\left(1 + \frac{ZY}{2}\right) + AZ\left(1 + \frac{ZY}{4}\right)$$
$$C_0 = c_1 + jc_2 = C\left(1 + \frac{ZY}{2}\right) + AY$$
$$D_0 = d_1 + jd_2 = A\left(1 + \frac{ZY}{2}\right) + CZ\left(1 + \frac{ZY}{4}\right)$$

TABLE LXXX—*Continued*

Transmission Line with Transformers at Each End:



$$A_0 = a_1 + ja_2 = A\left[\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + Z_sY_r\left(1 + \frac{Z_sY_s}{4}\right)\right]$$
$$+ BY_r\left(1 + \frac{Z_sY_s}{2}\right) + CZ_s\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{4}\right)$$

$$B_0 = b_1 + jb_2 = B\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + A\left[Z_r\left(1 + \frac{Z_rY_r}{4}\right)\left(1 + \frac{Z_sY_s}{2}\right)\right.$$
$$\left. + Z_s\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{4}\right)\right] + CZ_sZ_r\left(1 + \frac{Z_rY_r}{4}\right)\left(1 + \frac{Z_sY_s}{4}\right)$$

$$C_0 = c_1 + jc_2 = C\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + A\left[Y_r\left(1 + \frac{Z_sY_s}{2}\right)\right.$$
$$\left. + Y_s\left(1 + \frac{Z_rY_r}{2}\right)\right] + BY_rY_s$$

$$D_0 = d_1 + jd_2 = A\left[\left(1 + \frac{Z_rY_r}{2}\right)\left(1 + \frac{Z_sY_s}{2}\right) + Z_rY_s\left(1 + \frac{Z_rY_r}{4}\right)\right]$$
$$+ BY_s\left(1 + \frac{Z_rY_r}{2}\right) + CZ_r\left(1 + \frac{Z_rY}{4}\right)\left(1 + \frac{Z_sY_s}{2}\right)$$

4. Construct angle $\tan^{-1}\dfrac{(b_2)}{(b_1)}$.
5. Lay off I_rB_0 to the voltage scale, using the value of I_r corresponding to full-load, unity power factor.
6. Draw K_r circles about center M, using I_rB_0 for radius of 100 per cent circle and others in proportion.
7. Draw P_r lines perpendicular to I_rB_0 through points of intersection with K_r circles.
8. Draw PF_r lines radially from M, by laying off sines or tangents as preferred.
9. Draw E_s circles about center O, using the voltage scale.
10. Construct angle $\tan^{-1}\dfrac{(c_2)}{(c_1)}$.
11. Lay off E_rC_0 to a convenient current scale.
12. Construct angle $\tan^{-1}\dfrac{(d_2)}{(d_1)}$.
13. Lay off I_rD_0 to the current scale, using the value of I_r corresponding to full-load unity power factor.
14. Draw K_r circles, about center N, using I_rD_0 for radius of 100 per cent circle and others in proportion.

15. Draw P_r lines perpendicular to I_rD_0 through points of intersection with K_r circles.
16. Draw PF_r lines radially from N, by laying off sines or tangents as preferred.
17. Draw I_s circles about center O, using the current scale.
18. To use the diagram, select any load, P_r, and power factor, PF_r, and read E_s, I_s, and θ_s from which P_s and efficiency can be readily obtained by slide rule.

The Power Diagram is shown in Fig. 468. It is reproduced, with slight modifications, from the diagram developed by Evans and Sels in *The Electric Journal*, December, 1921, page 530. The accuracy of the diagram is limited only by the degree of care exercised in constructing and reading it, as it is based on the exact performance equations. The diagram may be constructed to show the performance of any form of network, by first computing the four general circuit constants, as explained in Sec. 436. It may be constructed in any one of four general forms shown at the bottom of Fig. 468, in order to show the particular performance required by the analysis. After the general circuit constants, A_0, B_0, C_0 and D_0, have been computed, the subsequent operations in constructing and reading the diagram are as follows:

1. Using a sheet of cross-section paper, lay off suitable scales of kw., kv-a. lead and kv-a. lag along the axes passing through O. The proper quadrants in which to locate the centers of the circles are determined by an inspection of the signs as explained below. The equation of a circle with center at point $+a$, $+b$, in the first quadrant is:

$$(x - a)^2 + (y - b)^2 = C^2. \qquad \qquad (227)$$

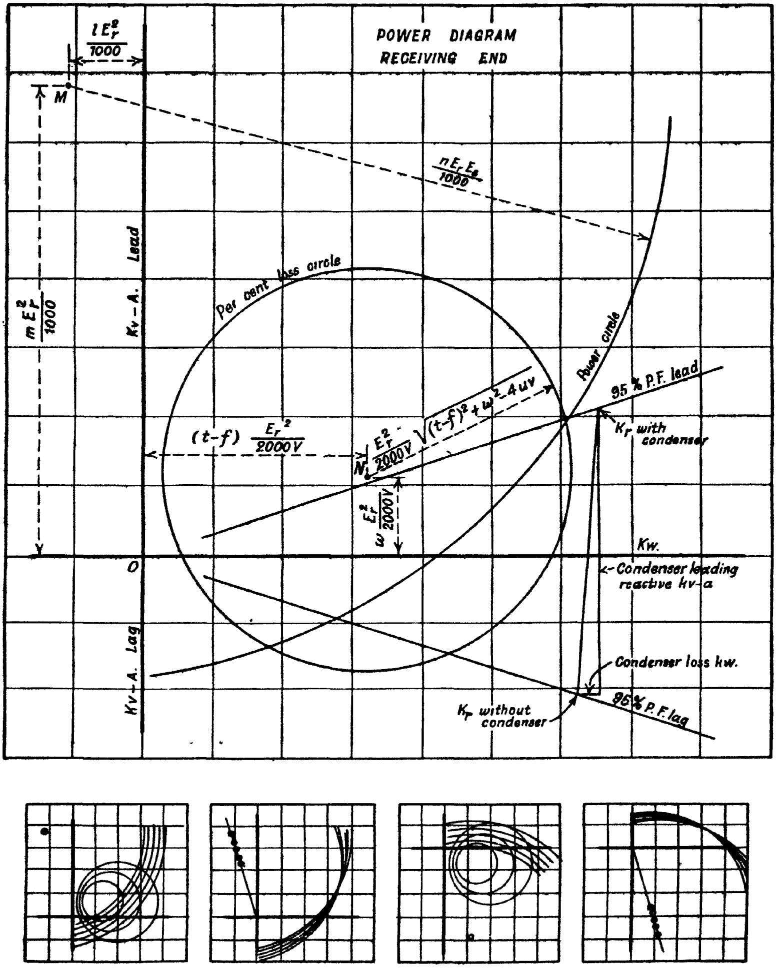
2. Draw PF lines radially from O by laying off sines or tangents as preferred.
3. Locate centers M and draw several power circles, using the power-circle equation given below and varying E_r or E_s as desired.
4. Locate centers N and draw several per cent loss circles, using the per-cent-loss-circle equation as given below and varying f as desired.
5. From the completed diagram, with fixed power, power factor, and voltage at one end of the system, the voltage at the other end and the per cent power loss may be read directly. The effect of a condenser at the receiving end can be obtained from the condenser triangle as shown on the diagram.

The notation and equations for power at the receiving end are as follows:

Notation

P_r = total three-phase power component at receiving end in kilowatts;

Q_r = total three-phase reactive component at receiving end in kilovolt-amperes. Q_r is positive for leading power factor and negative for lagging power factor.



Receiving End.

E_r constant
E_s various
f in percent of P_r

E_r various
E_s constant
f in percent of P_r

Sending End.

E_s constant
E_r various
f' in percent of P_s

E_s various
E_r constant
f' in percent of P_s

FIG. 468.

$K_r = P_r + jQ_r$ = total three-phase volt-amperes at receiving end;
E_r = line voltage at receiving end in volts;
E_s = line voltage at sending end in volts;

$$l = \frac{a_1b_1 + a_2b_2}{b_1^2 + b_2^2};$$

$$m = \frac{a_1b_2 - a_2b_1}{b_1^2 + b_2^2};$$

$$n = \frac{1}{\sqrt{b_1^2 + b_2^2}};$$

$$t = (a_1b_1 + b_1c_1 + a_2d_2 + b_2c_2 - 1);$$

$$u = (a_1c_1 + a_2c_2);$$

$$v = (b_1d_1 + b_2d_2);$$

$$w = (a_1d_2 + b_2c_1 - a_2d_1 - b_1c_2);$$

f = power loss in per cent of power at receiving end.

Power-circle Equation

$$\left(P_r + \frac{lE_r^2}{1000}\right)^2 + \left(Q_r - \frac{mE_r^2}{1000}\right)^2 = \left(\frac{nE_rE_s}{1000}\right)^2. \quad . \quad . \quad (228)$$

Constants l, m and n are always positive and, therefore, the center of the circle is in the second quadrant.

Per-cent-loss-circle Equation

$$\left[P_r + (t - f)\frac{E_r^2}{2000v}\right]^2 + \left[Q_r - w\frac{E_r^2}{2000v}\right]^2 = \left[\frac{E_r^2}{2000v}\sqrt{(t-f)^2 + w^2 - 4uv}\right]^2. \quad . \quad . \quad (229)$$

Constants u and v are always positive. Constants t and w may be positive or negative, depending on the system characteristics, but as f is always larger than t, $(t - f)$ is always negative. The center of the circle is, therefore, in the first quadrant if w is positive and in the fourth quadrant if w is negative.

Equation for Point of Maximum Effciency

$$f = \pm\sqrt{4uv - w^2} + t. \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (230)$$

The notation and equations for power at the sending end are as follows:

Notation

P_s = total three-phase power component at sending end, in kilowatts.

Q_s = total three-phase reactive component at sending end, in kilovolt-amperes. Q_s is positive for leading power factor and negative for lagging power factor.

$K_s = P_s + jQ_s$ = total three-phase volt-amperes at sending end;

E_s = line voltage at sending end, in volts;

E_r = line voltage at receiving end, in volts;

$$l' = \frac{d_1b_1 + d_2b_2}{b_1^2 + b_2^2};$$

$$m' = \frac{d_1b_2 - d_2b_1}{b_1^2 + b^{2}};$$

$$n = \frac{1}{\sqrt{b_1^2 + b_2^2}};$$

$$t = (a_1d_1 + b_1c_1 + a_2d_2 + b_2c_2 - 1);$$

$$u' = (c_1d_1 + c_2d_2);$$

$$v' = (a_1d_1 + a_2d_2);$$

$$w' = (a_2d_2 + b_2c_2 - a_1d_2 - b_1c_2;$$

f' = power loss in per cent of power at sending end.

Power-circle Equation

$$\left(P_s - \frac{l'E_s^2}{1000}\right)^2 + \left(Q_s + \frac{m'E_s^2}{1000}\right)^2 = \left(\frac{nE_sE_r}{1000}\right)^2. \quad . \quad . \quad (231)$$

Constants l', m' and n are always positive and, therefore, the center of the circle is in the fourth quadrant.

Per-cent-loss-circle Equation

$$\left[P_s - (t + f')\frac{E_s^2}{2000v'}\right]^2 + \left[Q_s + w'\frac{E_s^2}{2000v'}\right]^2$$
$$= \left[\frac{E_s^2}{2000v'}\sqrt{(t + f')^2 + w'^2 - 4u'v'}\right]^2. \quad . \quad (232)$$

Constants u' and v' are always positive. Constants t and w' may be positive or negative, depending on the system characteristics, but as f' is always larger than t, $(t + f')$ is always positive. The center of the circle is, therefore, in the first quadrant if w' is negative and in the fourth quadrant if w' is positive.

Equation for Point of Maximum Efficiency

$$f' = \pm\sqrt{4u'v' - w'^2} - t. \quad . \quad . \quad . \quad . \quad . \quad . \quad (233)$$

438. Structural Features.[1]—The type of construction to be adopted for a transmission line depends on the cost of construction as compared with the cost of maintenance, with due consideration given to losses from interruptions to service. Interruptions are often brought about through the action of the elements, high winds and sleet formation being the two worst offenders. It therefore follows that, if a line is to be constructed which will provide as nearly uninterrupted service as possible, general climatic conditions should be carefully studied and consideration given to local conditions prevailing on the whole or any part of the particular line.

439. Conductor Size and Material.—Before proceeding with the location and design of the towers, the tower dimensions must be determined. This

[1] This and following paragraphs on structural features of transmission lines have been prepared by H. O. Weber, Engineer, Stone & Webster, Inc.

involves determination of the characteristics of the conductors. Assuming that, after pursuing the economic considerations outlined in Sec. 431, there may be more than one type of conductor equally satisfactory from the standpoint of electrical service, an investigation should be made to determine the size and material for maximum structural economy. A conductor having a high tensile strength can be strung with less relative sag than one having a lower tensile strength, and a corresponding increase in the spacing of towers of the same height can be made. If the saving due to the decrease in the number of towers is enough to offset the increase in the tower weights due to the increased loading, the line with the longer spans will be more economical because of the saving in insulators and consequent saving in maintenance.

The total cost per mile of line, including conductors, insulators, towers, and foundations, should be estimated for each of the conductors under consideration. Care should be taken to be sure that the most economical spacing of towers is used in each case, as this spacing will be different for different conductors. Fig. 469 illustrates a convenient method of finding the most economical spacing of towers for a given conductor.

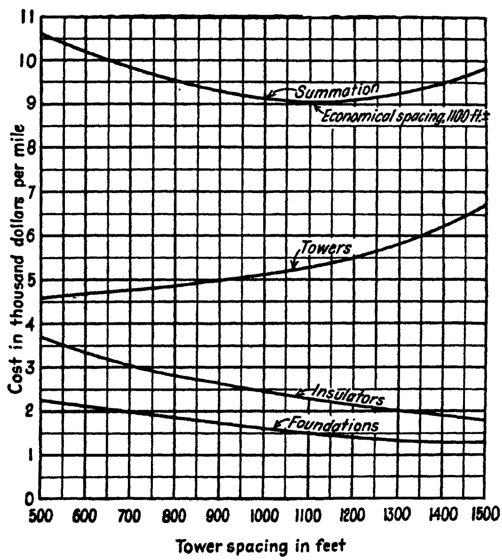


FIG. 469.—Determination of Economical Tower Spacing.

In determining the economical spacing it is not necessary to include items of cost that do not vary with the tower spacing. But, to make a comparison of line costs to determine the most economical conductor, the conductor costs should, of course, be included.

440. Conductor Loading.—The heights of towers and the loads to be sustained by them are dependent on the conductor sag and loading. Loadings recommended by the U. S. Bureau of Standards for various parts of the United States are shown in Fig. 470. The so-called Heavy Loading is the same as Class *B* loading recommended by the Committee on Overhead Line Construction of the National Electric Light Association and is the loading in most general use in the United States where sleet may occur.

The chart should not be used blindly but only as a guide, and the importance of service and amount of interruption permissible should be considered in determining the loads to be used. Lines located in regions where especially severe wind and sleet storms are encountered should be designed for a heavier

loading if interruptions to service are to be minimized. A wind pressure of 11 lb. per square foot on the projected area of the wire coated with a radial

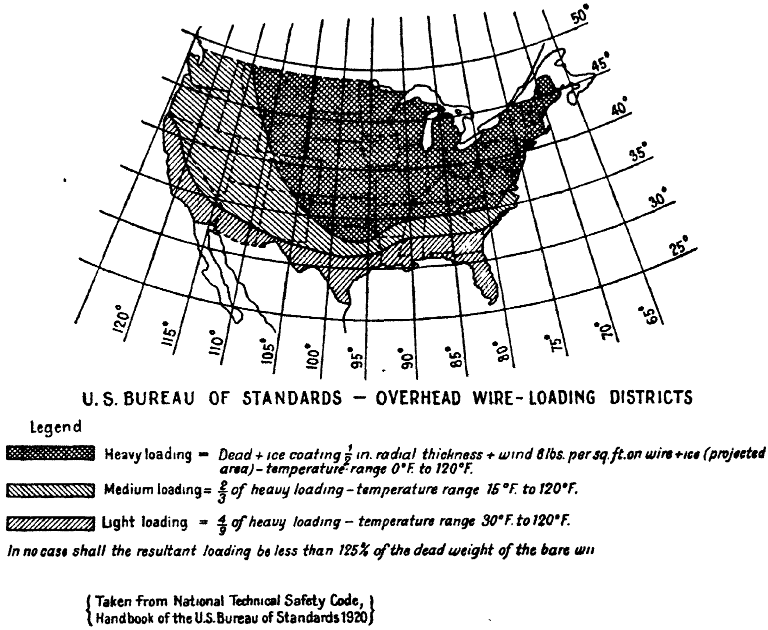


Fig. 470.—Loading Districts for Wire Spans Transmission Line Towers.

thickness of ¾ in. of ice, as specified by the above Committee for Class C loading, has been used in the design of some important lines where past experience shows that very heavy sleet formation occurs frequently.

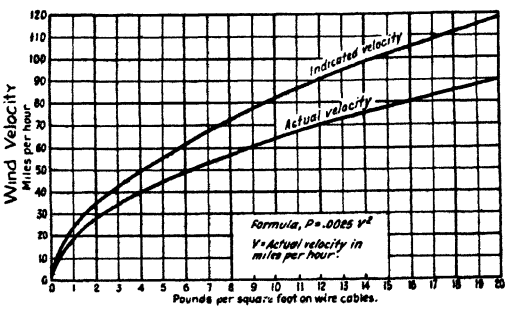


Fig. 471.—Wind Velocity and Pressure.

The wind pressure per square foot of projected area of a cylindrical wire is less than on a flat surface. Several formulae have been derived for the pressure on a wire interms of the wind velocity. Of these the one probably in most general use is·

$$P = 0.0025V^2, \quad \ldots \ldots \ldots \ldots \quad (234)$$

in which P is the pressure in pounds per square foot of projected area and V is the actual wind velocity in miles per hour.

The curves in Fig. 471 show pressures for various wind velocities and the

relation between indicated and actual velocities. From these curves it will be seen that a pressure of 8 lb. per square foot provides for an actual wind velocity of about 56.5 miles per hour, and that 11 lb. provides for about 66.3 miles per hour. A pressure of 15 lb. per square foot of projected area, corresponding to an actual velocity of about 77.5 miles per hour, is generally taken as the maximum for wires with no ice formation. For further discussion of this subject see "Mechanical Characteristics of Transmission Lines," by L. E. Imlay, *The Electric Journal*, January, 1925, page 3.

441. Conductor Sag.—A cable of uniform cross-section and material, perfectly flexible but inelastic, suspended at two points in the same horizontal plane and subjected only to its own weight, assumes the form of the common catenary. If the cable is of elastic material, it assumes the form of the elastic catenary. The maximum stress in the cable is at the point of support, and the minimum stress is at the lowest point. The stress at any point in the cable has two components: a horizontal component, which is uniform throughout the length of the cable; and a vertical component, which varies along each half of its length. The vertical component of the tension at any point is equal to one-half the weight of the cable in the span, less the weight of the cable between the point and the nearest point of support. The horizontal component of the tension at any point is equal to the minimum tension in the cable.

If it is assumed that the weight of the cable is distributed uniformly along a horizontal line instead of along the length of the cable, the equation for the curve assumed by the cable will be that of a parabola. There are, therefore, two general methods of calculating spans, the one being based on the elastic catenary, the other on the parabola. The results of the two methods of calculation will be almost identical when the sag is small, but the error due to the parabolic assumption becomes greater as the sag increases. For all practical purposes, the error is negligible for spans up to 1000 ft. and can generally be disregarded for spans up to 1500 ft. For spans up to this length, with wires so strung that the sag will not exceed one-tenth (1/10) of the span, the error will be less than 3 per cent.

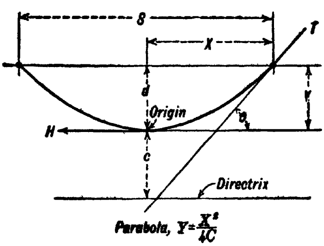


FIG. 472.

The following general equations are derived from the parabola (see Fig. 472).

$$\text{Sag in feet at any point} = \frac{WX^2}{2H} \quad \ldots \ldots \quad (235)$$

$$\text{Maximum sag, in feet} = d = \frac{WS^2}{8H} \quad \ldots \ldots \quad (236)$$

$$\text{Horizontal stress, in pounds} = H = \frac{WS^2}{8d} \quad \ldots \ldots \quad (237)$$

$$\text{Vertical stress, in pounds} \quad = V = \frac{W(3S^2 + 8d^2)}{6S}. \quad (238)$$

$$\text{Maximum stress, in pounds} \quad = T = \frac{WS}{8d}\sqrt{S^2 + 16d^2}. \quad (239)$$

$$\text{Length of cable, in feet} \quad = L = S + \frac{8d^2}{3S}. \quad . \quad . \quad (240)$$

$$\text{Maximum sag, in feet} \quad = d = \sqrt{\frac{3S(L - S)}{8}}. \quad . \quad . \quad (241)$$

$$\text{Change in length of cable, in feet} = e = \frac{TL}{AE} = tL\theta, \quad . \quad . \quad (242)$$

where W = weight, in pounds per foot of cable and load;
A = area of cross-section of cable, in square inches;
E = modulus of elasticity of cable;
θ = coefficient of expansion for cable;
X = horizontal distance, in feet, from any point in span to the nearest support;
S = length of span, in feet;
t = change of temperature, in degrees Fahrenheit.

442. Stress-deflection Curves.—As the cable is subject to changes in temperature there are four variables involved in the solution of the sag problem for any given span. They are length, tension, temperature, and sag, and all are closely interrelated. For example, a change in temperature causes a change in length; this causes a change in sag, which in turn changes the length and tension still further.

It is evident that a mathematical solution to find the sags and tensions for various conditions of loading and for changing temperatures would be very complicated. A combination of mathematical and graphical solutions very much simplifies the problem. If two sets of curves for a given span are drawn on the same sheet, one set showing the relation between sag and tension under various loadings, and the other showing the relation between sag and tension at various temperatures, their intersections will show sags and tensions for given loadings and at the different temperatures assumed. Such curves are known as stress-deflection curves and are illustrated in Fig. 473.

For convenience, the two systems of curves will be defined as follows:

(*a*) Sag-tension curves—showing the relation between sag and tension for a given span and loading.

(*b*) Pull-up curves—showing the relation between sag and tension for a given span and temperature.

The pull-up curve for any given span is a sag-tension curve at constant temperature and varying load, and is given this name to distinguish it from the sag-tension curve with a constant load. The term pull-up is derived from the method of procedure used in its construction, which consists of gradually

reducing the load on the cable at a constant temperature, thereby decreasing both the sag and tension, and " pulling-up " the cable.

The method of preparing these curves by use of the parabola equations is explained by the following problem:

Determine the maximum sag and side swing for a No. 4/0 B. & S. stranded hard-drawn copper conductor for a span of 800 ft., the maximum tension in the conductor not to exceed 5000 lb., the assumed loading to be ½ in. radial thickness of ice plus a wind pressure of 8 lb. per square foot on the projected area of the ice-covered cable, or a wind pressure of 15 lb. per square foot on the projected area of the bare cable, and the assumed temperature range to be from 0° to 120° F.

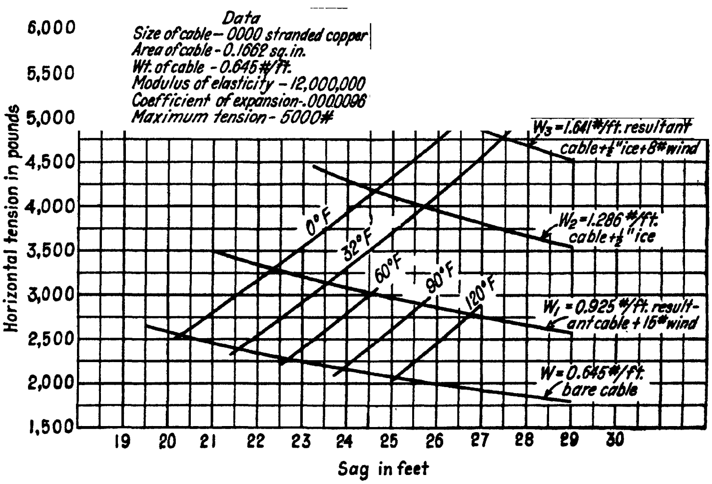


FIG. 473.—Stress Deflection Curves—800-ft. Span.

S = span—800 ft;

T = actual tension in cable (5000 lb. maximum);

H = horizontal stress in pounds at low point of cable;

A = cross-sectional area of cable = 0.1662 sq. in;

E = modulus of elasticity = 12,000,000;

θ = coefficient of linear expansion = 0.0000096;

W = 0.645 lb. per linear foot (weight of bare cable);

W_1 = 0.925 lb. per linear foot (resultant of cable weight and wind pressure);

W_2 = 1.286 lb. per linear foot (weight of cable plus ice);

W_3 = 1.641 lb. per linear foot (resultant of cable and ice weight and wind pressure);

EA = 1,995,000;

t = temperature change, in degrees Fahrenheit.

443. Sag-tension Curves.—Using Eq. (236), compute the sags for varying horizontal tensions for each of the loadings, W, W_1, W_2, and W_3 as follows:

Loading W		Loading W_1		Loading W_2		Loading W_3	
$d = \frac{0.645 \times 800^2}{8H} = \frac{51,600}{H}$		$d = \frac{0.925 \times 800^2}{8H} = \frac{74,000}{H}$		$d = \frac{1.286 \times 800^2}{8H} = \frac{102,880}{H}$		$d = \frac{1.641 \times 800^2}{8H} = \frac{131,280}{H}$	
W		W_1		W_2		W_3	
H	d	H	d	H	d	H	d
2600	19.9	3500	21.1	4200	24.5	5200	25.3
2400	21.5	3200	23.1	4000	25.7	5000	26.3
2200	23.5	3000	24.7	3700	27.8	4700	27.9
2000	25.8	2800	26.4				
1800	28.7	2600	28.5				

From the above values of H and d, plot the four curves W, W_1, W_2, and W_3 as shown in Fig. 473. These are the sag-tension curves.

Since the maximum tension will occur under the heaviest load and at the lowest temperature, the actual tension in this case should be limited to 5000 lb. under loading W_3. A maximum tension of 5000 lb. for this span and loading is equivalent to a horizontal tension of 4955 lb. which is the starting point for the pull-up curve at 0° F.

444. Pull-up Curves.—The sag under a horizontal tension of 4955 lb. and under loading W_3 is 26.45 ft. From Eq. (240) the length of the conductor under loading W_3, with a tension of 4955 lb. and a sag of 26.45 ft., is:

$$L = 800 + \frac{8 \times 26.45^2}{3 \times 800} = 802.325 \text{ ft.}$$

Now assume a reduction in load which will reduce the tension by some convenient decrement, as 500 lb., and compute the change in length of cable and new length for a number of such decrements. By Eq. (242) the decrease in length per 500 lb. reduction in tension is:

$$e = \frac{500 \times 802.325}{1,995,000} = 0.201 \text{ ft.}$$

Tension	4455	3955	3455	2955	2455
Original length	802.325	802.325	802.325	802.325	802.325
Reduction in length	.201	.402	.603	.804	1.005
New length	802.124	801.923	801.722	801.521	801.320

From these new lengths the corresponding sags are computed from Eq. (241) as follows:

For tension of 4455 lb., $d = \sqrt{\frac{3}{8} \times 800(802.124 - 800)} = 25.2$ ft.

For tension of 3955 lb., $d = \sqrt{\frac{3}{8} \times 800(801.923 - 800)} = 24.0$ ft.

For tension of 3455 lb., $d = \sqrt{\frac{3}{8} \times 800(801.722 - 800)} = 22.7$ ft.

For tension of 2955 lb., $d = \sqrt{\frac{3}{8} \times 800(801.521 - 800)} = 21.4$ ft.

For tension of 2455 lb., $d = \sqrt{\frac{3}{8} \times 800(801.320 - 800)} = 19.9$ ft.

For these values of sag and corresponding tension the 0° pull-up curve is drawn as shown in Fig. 473.

The next step is the determination of the changes in sag and tension as the temperature increases from its minimum of 0° F. to its maximum of 120° F. This is done by plotting additional pull-up curves between these temperature limits. Additional curves have been drawn for 32°, 60°, 90°, and 120° in Fig. 473 for this problem.

The starting point on the 32° pull-up curve is the length of the cable at 0° with a tension of 4995 lb., which was found to be 802.325 ft. With the loading unchanged, the increased length due to changing the temperature from 0° to 32° is found from Eq. (242) as follows:

Original length	= 802.325 ft.
Change in length, $e = 32 \times 802.325 \times 0.0000096$	= 0.246 ft.
New length at 32°	= 802.571 ft.

The corresponding sag from Eq. (241) is:

$$d = \sqrt{\tfrac{3}{8} \times 800(802.571 - 800)} = 27.8 \text{ ft.}$$

This is the sag corresponding to a tension of 4955 lb. at 32° and is a point on the 32° pull-up curve.

By reducing the tension by equal decrements, computing the change in length by Eq. (242) and finding the new lengths and corresponding sags as was done for the 0° pull-up curve, additional points are obtained from which the 32° pull-up curve is plotted.

The remaining curves are plotted in a similar manner.

Pull-up curves from 0° to 32° inclusive must be so drawn as to intersect all sag-tension curves, but pull-up curves for temperatures higher than 32° need not be drawn to intersect sag-tension curves for ice loading.

From the curves it is found that the maximum vertical sag is 25.7 ft. and occurs at a temperature of 32° F. under a loading of ½ in. ice and no wind, this sag being greater than the vertical components of the maximum sags under loadings including wind pressure. The maximum side swing is 18.9 ft. and occurs at a temperature of 120° F. with a 15-lb. wind on the bare cable.

The figures for maximum vertical sag and horizontal side swing for the most economical span should be used in determining the heights of towers and the normal width of right of way.

445. Catenary Solution.—If it is desired to compute the sags on the basis of the elastic catenary, they may be figured as described for the parabola and corrected by the percentage increase obtained from the curve in Fig. 474. For further information on mechanical features of design, see "Transmission Line Design, Part I: Mechanical Features," by F. K. Kirsten; and Part I, Sec. B, "Mechanical Design of Spans with Supports at Unequal Elevation," by G. S. Smith, University of Washington Engineering Experiment Station

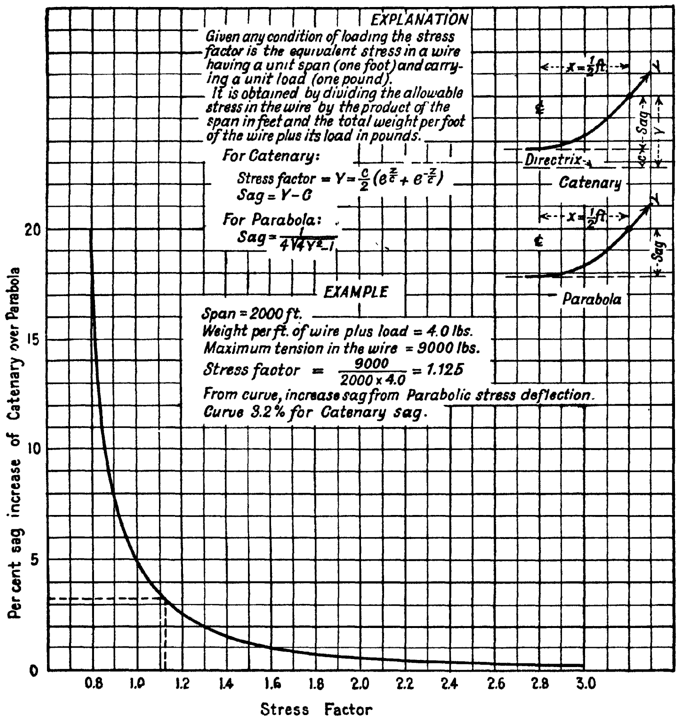


FIG. 474.—Comparison of Parabola and Catenary.

Bulletins No. 17, February, 1923, and No. 29, August, 1924, Seattle, Washington.

446. Stringing Curves.—It is essential to give the field forces information which will enable them to string the cables in such a way that under any of the assumed loading conditions the maximum tension cannot exceed the allowable. Probably the best way to present this information is by the use of stringing curves which show the relation between the sag and the span or the relation between the tension and the span at various temperatures.

Points for plotting these curves are obtained from the stress-deflection

curves for various spans and temperatures. Typical stringing curves are shown in Fig. 475.

447. Structures.—A decision as to the character of supports to be used should be governed by the general conditions outlined under Sections 426, 428, 438, 439, and 440. Wooden structures, either poles or H-frames, may be satisfactory where permanency is not essential. However, it is practically impossible to obtain as high a factor of safety as with steel structures, and, owing to the comparatively short spans which must be used because of the limitations in height, the number of insulators is greatly increased, thus proportionately increasing the probability of interruptions to service. For

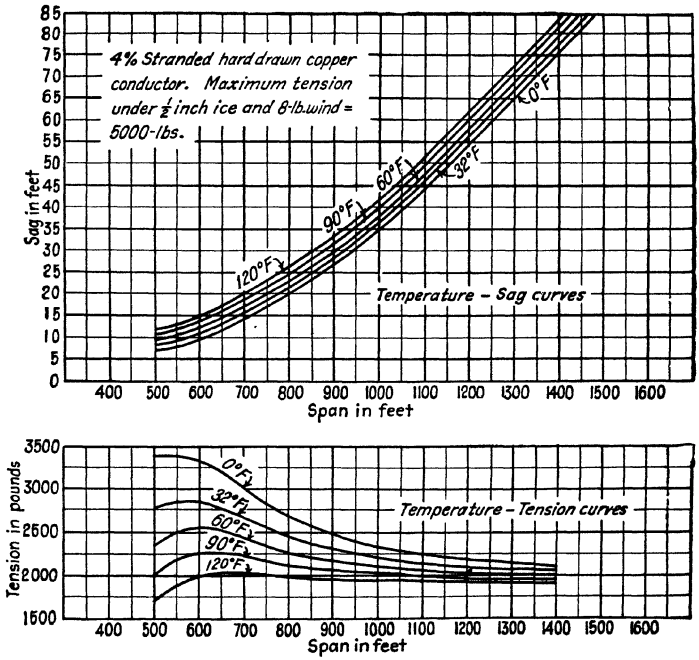


Fig. 475.—Stringing Curves.

further discussion of wood-pole construction, see "Pole and Tower Lines," by R. D. Coombs.

The more permanent types of supports may be divided into three classes: the steel pole, the flexible steel frame, and the rigid, wide-base steel tower.

Steel poles are generally used where space will not permit the use of frames or towers and wood poles are not desirable. They are usually intended merely to take care of vertical loads combined with horizontal loads across or at right angles to the direction of the line, and are rarely designed to take care of any load in the direction of the line. When designed for such longi-

tudinal loading from broken or unbalanced wire pull, they must do the work of a tower; but, because of the small dimensions at the base, they require a larger amount of steel and heavier foundations.

Flexible bents, commonly called A-frames, have been used where cheapness of construction was important. Like poles, their chief function is to carry vertical loads and loads across the line, and they depend on the cables to transmit longitudinal loads to heavier structures placed at regular intervals along the line. Their use is not recommended as their lack of resistance to load in the direction of the line makes the stringing of wires difficult, and in case of failure of one frame all frames between it and the nearest strain tower are generally pulled down. If light structures must be used, a square tower weighing very little more than a flexible frame can be designed to give the same strength across the line and considerable resistance to longitudinal loads.

Rigid towers provide the maximum security against interruptions to service caused by tower failure. Their strength allows the use of long spans with a consequent reduction in the number of insulators and in the probability of insulator failure. General practice has been to divide this class of structures into the three following types:

Suspension or line towers,
Angle towers,
Dead-end or anchor towers.

Suspension or line towers are used on tangents in the line and are designed to support the unbalanced pull due to one or more broken conductors, in addition to the load from wind and ice on the cables.

Angle towers, located at all angles in the line, are designed for the same loads carried by suspension towers and the additional transverse loads due to angles in the line.

Dead-end or anchor towers are designed to take the dead-end pull from any or all cables, together with the loads due to wind and ice on the cables.

Unless the number of angles in the line is large and the distance between angles is short, the saving made by the use of angle towers is small and the use of one less type of tower will probably justify the use of anchor towers at all angles.

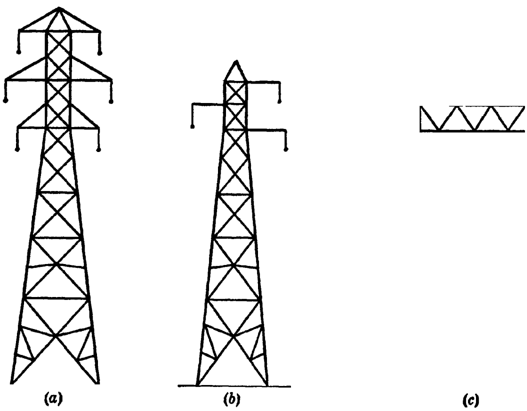
448. Tower Design.—From the standpoint of design, no definite line can be drawn between steel poles, flexible frames, and towers; and the present discussion will be limited to the type of structures generally called towers, having trussed framework, the corner posts being supported on separate foundations.

The outline and dimensions of the tower depend largely upon the arrangement and spacing of the conductors and the minimum clearance from the ground to the lowest conductor. The separation of conductors and clearances from conductors to tower members are determined almost entirely by electrical considerations. Clearance of cables from the ground is determined by the distance necessary to prevent accidents to individuals and interrup-

tions to service, and should not be less than that specified by the National Electrical Safety Code.

Where two three-phase circuits are carried on one set of towers, the conductors are usually arranged vertically as shown in Fig. 476*a*. The conductors of one circuit are on one side of the tower with the upper and lower ones in the same vertical plane, and the middle one offset outward from the tower. Where one circuit only is carried on a set of towers, they are either arranged in delta or in a horizontal plane as shown in Figs. 476*b* and 476*c*.

When the arrangement and spacing of conductors, together with the clearances from conductors to steel and to the ground, have been determined, the tower dimensions may be worked out and the design of the towers completed. The height from the ground to the bottom cross-arm of a suspension or line tower should equal the sum of the distance from ground to the low



Types of Towers

FIG. 476.

point of the conductor, the sag for the most economical span, and the vertical length of the insulator string. The dimensions of the tower at the base should be such as to give the most economical combination of tower and foundations.

The analysis of stresses and solution of problems involved in the details of design of transmission towers are not simple, and should be entrusted only to an experienced designer familiar with the calculation of stresses in framed structures. Great care should be taken to see that the combination of loads which produces a maximum stress is found for each tower member, as loads that produce the greatest stress in some members may not give maximum in others. Stresses should be computed for the following loads:

(*a*) Wind pressure on the tower applied at panel points of the tower.

(*b*) Pull from broken cables in the direction of the line, wind on the cables across the line, and vertical loads due to weight of cables and insulators plus ice coating, all applied at the points of attachment of cables.

(c) Pull across the line from angle in the line, applied at points of attachment of cables.

(d) Weight of the tower itself.

Graphical methods of analysis are generally used as they are sufficiently accurate and less laborious than mathematical solutions. Stress diagrams are drawn and the maximum combined stress in each member, for all loads, is found. The problem then consists in choosing appropriate structural sections to give the required strength, and connecting them in such a way that the strength of the members will be maintained.

449. Unit Stresses.—The unit stresses to be used in determining sizes of members in a transmission tower depend on the amount of overload that it is desired to have the towers capable of carrying. The following unit stresses are based on the results of a number of tower tests and tests of single angle pieces, and it is believed that towers designed in accordance with them will carry an overload of about 100 per cent if built from open-hearth steel which meets the requirements of the Standard Specification for Structural Steel, serial designation A-9 of the American Society for Testing Materials:

Tower Legs

Axial tension, net section............... 18,000 lb. per square inch;

Axial compression, gross section......... $20{,}000 - 85\,\frac{L}{R}$ lb. per square inch;

(For $\frac{L}{R}$ up to 150. Maximum 15,000.)

Web Members

Axial tension, net section............... 16,000 lb. per square inch;

Axial compression, gross section......... $18{,}000 - 80\,\frac{L}{R}$ lb. per square inch;

(For $\frac{L}{R}$ up to 150. Maximum 14,000.)

Axial compression, gross section......... $13{,}500 - 50\,\frac{L}{R}$ lb. per square inch;

(For $\frac{L}{R}$ up to 200.)

Bolts and Rivets

Shear............................. 15,000 lb. per square inch;

Bearing........................... 30,000 lb. per square inch.

Lower unit stresses are given for tower web members than for the leg members, because for towers of ordinary size the web members generally consist of single angles connected through one leg only. In case towers are of such size that these members can be conveniently connected through both legs, unit stresses specified for tower legs can be used for web members also.

For further discussion of the subject of the design of transmission towers, including structures, loading, and unit stresses, see the American Bridge Company's handbook "Transmission Towers," copyrighted 1925.

450. Location of Towers.—Suitable location of towers on the profile can best be determined by sliding a template of the maximum sag curve along the profile until the desired position of the span is found. The template should be made of transparent material, such as celluloid, and should show, in addition

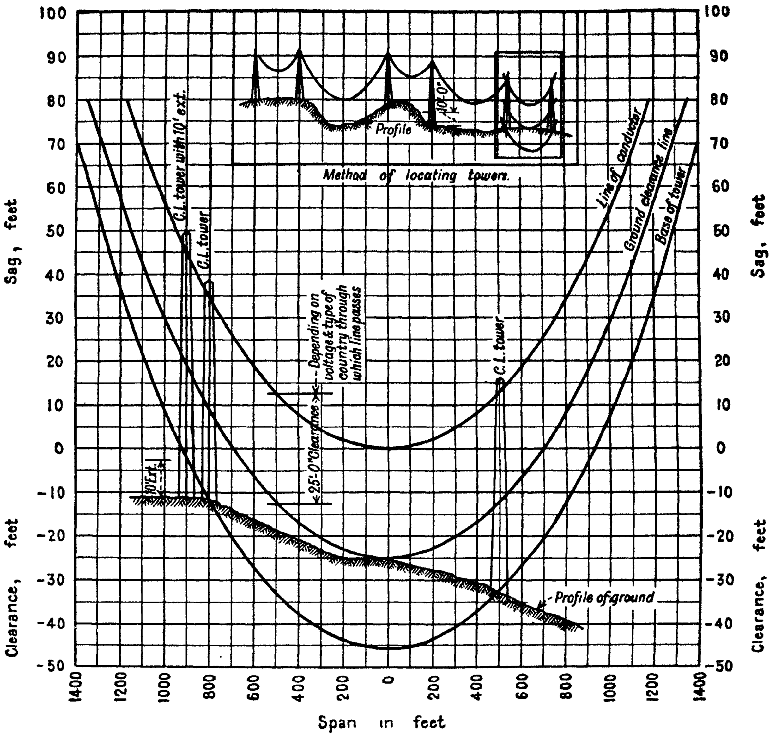


FIG. 477.—Tower Location Diagram.

to the sag curve, minimum clearance to ground and lengths of spans all drawn to the same scale as the profile.

Figure 477 shows curves for making such a template and a section of profile illustrating its use. It is well to show a minimum sag curve on the template and so locate towers that there will be no uplift on them under minimum sag conditions. When uplift cannot be avoided, the towers must be made to resist it.

It often happens that towers somewhat higher than those used for normal spans are desirable at certain locations to give the most economical arrange-

ment. To meet these requirements, tower extensions are provided so that the towers can be made of a standard height.

451. Protective Coating.—It is practically impossible to paint the portions of a tower above the level of the lowest conductors without deenergizing the line, and it has generally been found more satisfactory to use a galvanized structure. This has led to the use of bolted towers, as satisfactory galvanizing cannot be done on riveted work and riveting after the galvanizing has been done destroys the galvanizing near the rivets.

For special high towers with very heavy loading, where bolted joints would not be satisfactory, it is possible to rivet and paint the part of the towers below the conductors and galvanize the portion above.

Specifications for galvanizing should be rigid, and all galvanizing should be given very careful inspection.

452. Tower Foundations.—In general, there are two types of tower foundations, concrete anchors and steel anchors. The individual anchors for each leg of a tower should be designed to resist the maximum uplift or compression in the leg plus that from the possible overload.

A steel anchor consists of a structural member or members extending into the ground and connected to a steel grillage. The grillage should be of sufficient size to transmit the maximum compression to the soil without exceeding allowable bearing pressure, and should be buried to such a depth that it will resist the uplift from the tower leg plus any desired overload. In general, the resistance to uplift may be assumed equal to the weight of earth of an inverted frustum of a pyramid with a bottom area equal to the area of the grillage, its sides forming angles of 30 degrees with a vertical plane and its height equal to the depth from the surface of the ground to the grillage.

Concrete anchors should be designed to engage the surrounding earth so that the earth will resist uplift in combination with the weight of the concrete.

453. Outdoor Station Structures.—Another type of structure, in the same general class with steel transmission towers, is the steel structure used for supporting buses and switches for outdoor substations and switching stations.

The general arrangement and dimensions of these structures are usually determined largely by electrical considerations. The loads which they must support are those produced by the incoming or outgoing lines, the pulls from suspended buses, dead loads from switches, insulators, and structures, and wind on structure and equipment. Sleet formation should be given the same consideration as for the transmission line.

Some of the less important low-voltage structures are painted, but, because of the inconvenience of repainting, galvanized steel is generally used for high-voltage structures.

454. Bibliography.—This bibliography is applicable to Chapters XXIX to XXXII inclusive.

1. Alternating Currents, by C. E. Magnusson. McGraw-Hill Book Co., Inc., New York.
2. Engineering Mathematics, by C. P. Steinmetz. McGraw-Hill Book Co., Inc., New York.
3. Standard Handbook for Electrical Enginners. McGraw-Hill Book Co., Inc., New York.

4. Handbook for Electrical Engineers. Harold Pender, Editor-in-Chief. John Wiley & Sons, Inc., New York.
5. The Engineers' Manual, by R. G. Hudson. John Wiley & Sons, Inc., New York.
6. Standards of the A.I.E.E. American Institute of Electrical Engineers, New York.
7. Principles of Alternating-current Machinery, by R. R. Lawrence. McGraw-Hill Book Co., Inc., New York.
8. Alternating Currents and Alternating-current Machinery, by D. C. Jackson and J. P. Jackson. The Macmillan Co., New York.
9. Hydro-electric Power Stations, by D. B. Rushmore and E. A. Lof. John Wiley & Sons, Inc., New York.
10. The Electric Power Club Handbook of Standards. The Electric Power Club, Cleveland, Ohio.
11. National Electrical Safety Code. U. S. Bureau of Standards, Washington, D. C.
12. Electrical Characteristics of Transmission Circuits, by William Nesbit. Westinghouse Electric & Mfg. Co., East Pittsburgh, Pa.
13. Handbook for Overhead Line Construction. National Electric Light Association. Franklin Printing Co., Philadelphia.
14. National Electrical Safety Code. Dept. of Commerce, Bureau of Standards, Government Printing Office, Washington, D. C.

CHAPTER XXXIII

INVESTIGATIONS AND REPORTS

BY WILLIAM P. CREAGER

455. Purpose of Reports.—Engineers are frequently called upon to prepare reports on water-power sites, projects, or partially or wholly completed developments or systems of developments, to show their value under certain conditions and for certain purposes. The usual purposes for which reports of this type are required are as follows:

1. *Promotion.*—To assist in the promotion of a project or to furnish engineering advice to a prospective investor in a project.
2. *Marketing a Site, Plant, or System.*—To assist in the sale of a site, plant or system or to furnish engineering advice to a prospective purchaser of same.
3. *Consolidations.*—To estimate the value of the physical property of a system, for purposes of stock allotment in proposed consolidations of several going concerns.
4. *Choice of Site.*—To determine, for an owner of several sites, the best site to develop for certain market requirements.
5. *Condemnation.*—To estimate the value of a site, plant, or system in connection with condemnation proceedings.
6. *Physical Value.*—To estimate the present or replacement value of the physical property of a plant or system.

In all cases the essential feature to be shown by the report is the *value* of the site, plant, or system. Except in the case of reports on physical value, Item 6, the value is measured by the rate of return on the investment.

456. Extent of Report.—In most cases it is advisable to prepare a preliminary report for the perusal of the client, based on available data, a cursory examination of the site, and approximate estimates. If the preliminary report shows a value which is satisfactory to the client, it is superseded by a final report based on extended investigations and studies for a more accurate estimate of value.

The client may desire the final report to be simply an expression of opinion from the engineer, which could be set forth in a very brief summary. On the other hand, the writer has sometimes been required to give in his report a complete record of his data and studies, including tabulations of water supply and storage calculations, sketches of proposed structures, and details of the estimates of cost. These are, of course, the extremes, and the latter type of report is not frequently required as the engineer is expected to preserve his data and calculations for further use if needed.

457. Arrangement and Wording of Report.—The engineer, in his report, should not use technical terms that are not known to those for whom the report is intended, unless such terms are fully explained. Reports to laymen should differ materially in this respect from those to an engineer.

While some reports are written in the form of a memorandum, the most accepted form is that of a letter to the client.

The report should be introduced by a statement of its purpose and scope. Usually, in engineering reports, this is followed by a brief summary of conclusions, after which are given, in as much detail as required, a logical discussion of the various factors involved, and a description of the investigations and studies of the engineer. Maps, drawings, diagrams, lengthy tabulations, and similar exhibits are usually included in an appendix.

Neatness is an essential feature of a good report, as a poorly typed and bound report invariably carries with it an impression of inaccuracy. If the report is lengthy, it should be divided into sections, each with an appropriate side heading, and should preferably be accompanied by an index.

Table XC contains a combined outline of the essential data required for a final report on a plant or a project. What part of these data should be included in the report depends upon the desires of the client.

The following are typical headings of sections in a complete report on a project. These can be further subdivided as shown in Table XC.

(1) Purpose and scope of report;
(2) Summary of conclusions;
(3) Corporate history of existing company;
(4) Charters, franchises, and other assets;
(5) Physical features of the watershed;
(6) Water supply;
(7) Head available;
(8) Power available;
(9) Power market;
(10) Description of the site and existing property;
(11) Proposed new work and extensions;
(12) Description of field investigations;
(13) Estimate of money required;
(14) Estimate of gross annual income;
(15) Estimate of annual charges;
(16) Financial statement showing the value of the project.

Items 13, 14, and 15 are used in compiling the financial statement of Item 16. Typical financial statements for the several purposes for which reports are required are shown in the following sections.

458. Promotion Reports.—The promoter who has obtained control of a valuable water-power prospect, by purchasing or securing options on the water rights, flowage areas, dam sites, or other controlling features, seeks to realize a profit by the promotion of a new company to develop the site and sell the output. An engineer is engaged to write a report on the project, and this report is submitted to the bankers or others who are expected to under-

write the securities. Frequently the prospective underwriters also employ an engineer to make an independent examination and report. A typical financial statement for a promotion report is given in Table LXXXI.

The net income after operation (Item 20) should be sufficient to pay all interest charges (Item 21) and leave a satisfactory surplus (Item 27). In this example, the interest charges are assumed at 7 per cent of the total investment of $2,500,000, of which $300,000 is the amount the promoter expects to receive as compensation for his holdings.

TABLE LXXXI

TYPICAL FINANCIAL STATEMENT FOR A PROMOTION REPORT

Estimated Results of Operation in an Average Year

1.	Income			$365,000
2.	Delivered Primary Energy, 34,000,000 kw.-hr.@10 mills		$340,000	
3.	Delivered Secondary Energy, 5,000,000 kw.-hr.@5 mills		25,000	
4.	Operating Charges			102,500
5.	Operators		10,000	
6.	Power-development Operators	$7,000		
7.	Transmission-line Patrol	2,000		
8.	Substation Operators	1,000		
9.	Maintenance and Repairs		3,500	
10.	Power Development	2,000		
11.	Transmission Line	1,000		
12.	Substations	500		
13.	Taxes		12,000	
14.	Insurance		1,000	
15.	Depreciation Reserve		21,000	
16.	Management, Supervision and Home Office Expense		15,000	
17.	Other Miscellaneous Charges		5,000	
18.	Miscellaneous Credits		(5,000)	
19.	Steam Standby Charges, 4,000,000 kw.-hr. @ 10 mills		40,000	
20.	Net Income after Operation			262,500
21.	Interest Charges			
22.	On $300,000 cost of lands and water rights			
23.	On $1,500,000 cost of water-power development			
24.	On 200,000 cost of transmission and substations			
25.	On 500,000 cost of steam standby plant			
26.	On 2,500,000 total investment at 7 per cent			175,000
27.	Surplus Earnings			$87,500

28. Ratio of Net Income after Operation to Interest Charges $= \frac{262,500}{175,000} = 1.5$

29. Rate of Return on Investment $\frac{262,500 \times 100}{2,500,000}$ 10.5 per cent.

It is usual to show in Item 26 the total money required to finance the project and not the face value of the securities issued for that purpose. In the example, it has been assumed that 6.5 per cent bonds (or other securities) can be sold to net 93 per cent of their face value, including discount and selling expense. In order to obtain the $2,500,000 cash required, it would be neces-

sary to issue $\frac{2,500,000}{0.93}$ = \$2,690,000 face value of securities, and the equivalent interest rate on the cash received would be $\frac{6.5}{0.93}$ = 7 per cent, as used in the example.

The Ratio of Net Income after Operation, to Interest Charges shown in Item 28, is the index of value or factor of safety of the project. Or, as is frequently stated, "the project is estimated to earn enough to pay 1.5 times interest requirements." In order to make the project attractive to investors, this ratio must generally be between 1.5 and 2.5, depending upon the hazards incident to the project. If the promoter is willing to take common stock for his interests in the lands and water rights, that amount may be omitted from the investment used to obtain the ratio.

The ratio of Item 28 is also equal to the Rate of Return on the Investment of Item 29, divided by the interest rate, or $\frac{10.5}{7}$ = 1.5.

459. Report for Marketing a Site.—Instead of promoting a company to develop the site which he owns, the promoter may endeavor to sell his lands and water rights to a going concern. In this case, the financial statement may be made up in the form shown in Table LXXXI, which gives the rate of return on the investment, corresponding to a fixed price (Item 22) to be paid the promoter for the lands and water rights; or it may be made up in the form shown in Table LXXXII, which shows the value of the lands and water rights corresponding to a fixed desired minimum rate of return on the investment required for the development.

TABLE LXXXII

TYPICAL FINANCIAL STATEMENT SHOWING VALUE OF LANDS AND WATER RIGHTS

Estimated Results of Operation in an Average Year

1. Income (Same as in Table LXXXI)......................		\$365,000
2. Operating Charges (Same as in Table LXXXI)............		102,500
3. Net Income after Operation		\$262,500
4. Item 3 Capitalized at 10.5 per cent.....................	\$2,500,000	
5. Cost of Development, Transmission, Substations and Steam Standby (Items 23, 24 and 25 of Table LXXXI)........	2,200,000	
6. Value of Lands and Water Rights.......................	\$300,000	

The Net Income after Operation of Item 3, capitalized in Item 4 at the desired minimum rate of return[1] of 10.5 per cent, indicates the total possible capitalization of the project, from which the estimated cost of the development must be subtracted to indicate the value of the lands and water rights which are for sale.

This type of statement is used both by the vendor and by the prospective purchaser.

[1] See Sec. 465 for definition of "desired minimum rate of return."

460. Report for Marketing a Plant or System.—Table LXXXIII shows a typical financial statement of the form generally used to show the value of a plant or system which is for sale. It is of the same type as that shown in Table LXXXII for the sale of a site; but includes the additional features of existing structures.

TABLE LXXXIII

TYPICAL FINANCIAL STATEMENT SHOWING VALUE OF AN EXISTING DEVELOPMENT

Estimated Results of Operation in an Average Year

1.	Income			$390,000
2.	Sale of Output (Same as in Table LXXXI)		$365,000	
3.	Other Income		25,000	
4.	Operating Charges (Same as in Table LXXXI)			102,500
5.	Net Income after Operation			$287,500
6.	Item 5 Capitalized at 10.5 per cent		$2,730,000	
7.	Accounts Receivable and Other Assets[2]		70,000	
8.	Gross Value		$2,800,000	
9.	Deductions		1,800,000	
10.	Replacements Required at Once	$400,000		
11.	Accrued Depreciation on the Balance of Equipment of Structures	300,000		
12.	Required Expenditures for New Structures and Other Work	100,000		
13.	Accounts Payable, Mortgages, and Other Liabilities	1,000,000		
14.	Value of the Development.		$1,000,000	

The total income from a going concern, as shown in Item 1, is the estimated income from power sales plus other income, such as rentals, transmission tolls, miscellaneous sales, etc.

The records of income and operating charges, as shown by the company's books, may be used instead of an independent estimate by the engineer, provided the period of operation is indicative of normal conditions.[3]

The net income after operation of Item 5, capitalized in Item 6 at the desired minimum rate of return of 10.5 per cent, plus accounts receivable and other assets to be included in the transfer, indicates in Item 8 the gross value. This item is the total possible capitalization based on new unencumbered structures which are adequate for the use intended.

From this must be deducted the estimated cost of all replacements of worn-out structures and equipment that must be made at once; the accrued depreciation (Sec. 474) on the balance of the structures and equipment; the estimated cost of new structures and equipment for extension, expansion, steam standbys, transmission, and other things required to realize the estimated

[2] These must be valid and quick assets, capable of earning the same rate on the investment for their purchase as that for the purchase of the development as a whole.

[3] The financial statement of Table LXXXIII is set up, as indicated in the heading, for an average year. For cases where the statement is set up for worst conditions as explained later, such records, if used, should correspond to worst conditions.

income; and all accounts payable, mortgages, and other liabilities that are to be taken over.

The remainder, shown in Item 14, is the value of the development, including the lands, water rights, and assets and liabilities. As further explained in Sec. 474, the accrued depreciation shown in Item 11 represents a reserve fund which must be provided at once and to which future annual depreciation reserves (Item 15 of Table LXXXI) must be added each year to provide sufficient funds for replacement at the expiration of the life of the structures and apparatus.

It is to be noted that the original, the present, or the replacement value of the structures and equipment has no bearing on the sale value of the development; but the present value (Sec. 464) is frequently estimated and deducted from the total value of Item 14 to show the theoretical value of the lands and water rights.

If the sale price has been definitely fixed by the vendor, a financial statement of the type shown in Table LXXXIV may be used to show the probable return on the investment corresponding to such sale price. This table is of the same form as Table LXXXI described in Sec. 458.

TABLE LXXXIV

Typical Financial Statement Showing Return from Purchase of Existing Development

Estimated Results of Operation in an Average Year

1.	Income (From Table LXXXIII)			$390,000
2.	Operating Charges (From Table LXXXIII)			102,500
3.	Net Income after Operation			$287,500
4.	Interest Charges:			
5.	On	$1,000,000	purchase price of development,	
6.	On	400,000	purchase price of replacments required at once,	
7.	On	300,000	fund required for accrued depreciation as in Table LXXXIII,	
8.	On	100,000	cost of new work as in Table LXXXIII,	
9.	On	1,000,000	fund for liabilities as in Table LXXXIII,	
10.		2,800,000		
11.	Less	70,000	of accounts receivable and other assets[4]	
12.	On	2,730,000	total investment at 7 per cent	191,100
13.	Surplus Earnings			$96,400

28. Ratio of Net Income after Operation to Interest Charges. $\frac{287,500}{191,100} = 1.5.$

29. Rate of Return on Investment $\frac{287,500 \times 100}{2,730,000} = 10.5$ per cent.

461. Reports for Consolidations.—In the case of consolidations of two or more systems, the allotment of stock of the new corporation to the different systems is made from a comparison of their probable future earnings for stock dividends. The probable future earnings may be predicated on past earnings adjusted for the probable growth of the market and the probable earnings of owned or controlled undeveloped powers.

[4] See footnote for Table LXXXIII.

The probable earnings of the undeveloped powers may be shown by a financial statement similar to that indicated in Table LXXXI, described in Sec. 458, in which the surplus earnings of Item 27 are available for stock dividends. The costs to be included in the interest charges of Item 21 should include no past expenditures. Any particular project that did not show a satisfactory ratio of income to interest charges (Item 28) would of course be eliminated from consideration, no matter what the surplus earnings might be. It is evident that, of two projects having exactly the same earnings, the one with the greatest ratio of income to interest charges would be the better project; but no means has been found to evaluate satisfactorily the factor of safety in this respect.

If it is desired to disregard the statement of past earnings as shown in the system's accounts, the probable future earnings of the existing developments may be shown by a financial statement similar to that indicated in Table LXXXIV in which the purchase price of Item 5 would not be included.

462. Reports for Choice of Site.—Frequently a corporation owning a number of water-power sites desires to determine the one best suited to its new market requirements. Usually many of the sites may be eliminated by certain features which are unsuitable to the desired use, such as load-factor limitations, the capacity and output, the class of power, and similar items, and the remaining possibilities are compared by their respective earning capacities. The choice is made by a comparison of financial statements set up in one of the following two ways:

1. The probable return on the required additional investment may be indicated as shown in Table LXXXI.
2. The unit cost of producing energy may be shown by a financial statement similar to that shown in Table LXXXV.

Whichever method is adopted, the interest charges should *not* include those on money already invested. Such investments, whether great or small, have no bearing on the comparative values of the sites, as interest charges on past investments must be made whether the site is developed or not. The problem is to determine that site which will give the greatest benefit from *future investments*.

Table LXXXV, while sufficient for comparative purposes, gives no indication of the absolute value of the project except that the maximum cost of producing energy consistent with a satisfactory profit is usually known.

463. Reports for Condemnation Proceedings.—Where a corporation has the right of condemnation of power sites or developments for the purpose of expansion of its own system, the value of such sites or developments is usually shown by a financial statement of the type indicated in Table LXXXII or LXXXIII.

464. Reports on Physical Value.—Physical values may be required for establishing capitalization, for rate cases and for other similar purposes. The value for such purposes may be stated in one of the following ways, depending upon the requirements of the commission having jurisdiction in such matters.

TABLE LXXXV

TYPICAL FINANCIAL STATEMENT SHOWING COST OF PRODUCING ENERGY

Estimated Results of Operation in an Average Year

1.	Operating Charges (Same as in Table LXXXI)	$102,500
2.	Interest Charges on Future Expenditures,	
3.	On $300,000 cost of additional lands and water rights required,	
4.	On 1,500,000 cost of water power development,	
5.	On 200,000 cost of transmission and substations,	
6.	On 500,000 cost of steam standby plant,	
7.	On 2,500,000 total additional investment at 7 per cent	175,000
8.	Total Cost of Producing Power	$277,500
9.	Total Output,	
10.	Delivered Primary Energy, 34,000,000 kw.-hr.	
11.	Delivered Secondary Energy, 5,000,000 kw.-hr.	
12.	Total Delivered Energy 39,000,000 kw.-hr.	
13.	Cost per kw.-hr. of Total Delivered Energy, $\frac{277{,}500 \times 100}{39{,}000{,}000} = 7.12$ mills.	
14.	Cost per kw.-hr. of Equivalent Primary Energy (based on secondary energy being worth one-half of primary), $\frac{277{,}500 \times 100}{34{,}000{,}000 + \frac{5{,}000{,}000}{2}} = 7.60$ mills.	

1. *The Investment Value.*—The actual amount invested in the development less accrued depreciation. The actual investment would include the purchase price of lands and water rights, the original cost of structures and equipment and the actual cost of betterments.

2. *The Replacement Value.*—An estimate of what the cost would have been if developed just prior to the date of the report plus a reasonable amount for the value of the lands and water rights.

3. *The Present Value.*—The replacement value less accrued depreciation.

For capitalization purposes, the investment value is frequently required, although it would seem that the assets of a corporation should include the possible sale price of the development as indicated by a financial return similar to that shown in Table LXXXIII.

The replacement value is frequently adopted by commissions having jurisdiction over public service corporations as a basis for fixing rates to be charged for energy.

The present value has been used for establishing taxation, and represents the actual physical value at the date of the report.

A reasonable value of lands and water rights, as used in replacement and present values has been very difficult to define. There has been no standardization in this respect and the engineer must be guided by the usual procedure in the district where the report is to be used.

Accrued depreciation is explained in Sec. 474.

465. Desired Minimum Rate of Return.—It should be borne in mind that the "desired minimum rate of return," as used in the foregoing financial statements, must conform to the following criteria.

1. It must correspond to a satisfactory rate of return for that type of investment.
2. It must be at least equal to the return which could be obtained from any possible alternative source of power suited to the market requirements.

If these two criteria are followed, the resulting value will be the same for all of the many rational variations of the foregoing financial statements which have been proposed and used, including the " Competitive Plant Method " described in Sec. 467.

It is evident that the value of the lands and water rights shown in Table LXXXIII would not be a rational one, even when the 10.5 per cent return is satisfactory, if a steam plant could be built for the same market requirements to earn a greater percentage return. Not only would the prospective investor hesitate to consider such a project when a more attractive one was available, but he would be afraid of competition.

The alternative source of power in Criterion 2 may be a steam plant, an internal combustion plant, another water-power project or the possibility of purchasing power from another system, requiring an investment in transmission and distribution only.

If the only alternative source of power is not a steam plant or other type of plant that can be extended in capacity indefinitely, but is, for instance, a single water power which has capacity to take care of only a few years of market growth, the alternative source can be considered as affecting only the present value of the project and not its future value. Therefore, if 10.5 per cent is considered a satisfactory minimum rate of return but the alternative could earn 15 per cent, the 10.5 per cent would be used in such cases, provided interest and taxes during the years of idleness before the development is made are included in the cost of development as explained in Sec. 466.

466. Values for Future Development.—If a project is not suitable for immediate needs or if its development must be postponed because there are more favorable projects suitable for immediate market requirements, it may be purchased for use sometime in the near future. In this case the total amount invested in lands and water rights at the end of the period of idleness will be the sum of the following items.

(*a*) The amount paid for the lands and water rights,
(*b*) The interest on the purchase price during the period,
(*c*) Taxes on the value of the lands and water rights during the period (value usually assumed equal to purchase price),
(*d*) Interest on the money borrowed to pay the interest and taxes.

In other words, the interest and taxes will be compounded during the period.

Let n = the period of idleness in years;
i = the interest rate, expressed as a decimal;
t = the tax rate, expressed as a decimal;
P = the principal amount paid for lands and water rights;
C = the total investment at the end of the period.

Then, if interest is paid quarterly

$$C = P\left(1 + \frac{i+t}{4}\right)^{4n}. \quad \ldots\ldots\ldots \quad (243)$$

And the compounded interest and taxes during the period of idleness is

$$I = P\left[\left(1 + \frac{i+t}{4}\right)^{4n} - 1\right]. \quad \ldots\ldots \quad (244)$$

The value of I should be included in the estimated cost of the development. (Item XXXI of Table XCI.)

467. Competitive Plant Method of Valuation.—This method has been adopted extensively and, if correctly used, will give values in accordance with the methods previously described; but very frequently it has been incorrectly used and has led to absurd results.

Table LXXXII shows a case where certain lands and water rights were computed to be worth \$300,000, this value being based on a desired minimum rate of return of 10.5 per cent. In other words, if \$300,000 were paid for the lands and water rights, the project could be developed and earn 10.5 per cent on the total investment. We shall now derive the value, first by a frequently used but erroneous application of the competitive plant method and, second, by the correct application.

Let the alternative source of power be a steam plant capable of earnings as indicated in Table LXXXVI.

TABLE LXXXVI

ABBREVIATED FINANCIAL STATEMENT FOR THE ALTERNATIVE STEAM PLANT

1. Income (Same as Table LXXXII)................................		\$365,000
2. Operating Charges..............................		197,000
3. Net Income after Operation..		168,000
4. Investment...	\$1,600,000	
5. Rate of Return on Investment, $\frac{168{,}000 \times 100}{1{,}600{,}000}$ = 10.5 per cent.		

The incorrect method of stating the competitive plant method is as follows:

If the total cost of producing energy from a project, excluding interest charges on the purchase price of lands, water rights, and existing structures, is less than the total cost of producing the equivalent energy from the alternative source; then the difference, or saving in total cost of producing energy, capitalized at a satisfactory rate of interest, is equivalent to the value of such lands, water rights, and existing structures.

This erroneous rule would give the following value of the lands and water rights valued in Table LXXXII at \$300,000.

TABLE LXXXVII

ABBREVIATED FINANCIAL STATEMENT SHOWING VALUE OF LANDS AND WATER RIGHTS BY ERRONEOUS APPLICATION OF COMPETITIVE PLANT METHOD

Cost of producing energy from the project exclusive of interest on lands and water rights		$256,500
Operation (From Table LXXXII)	$102,500	
Interest on Construction Cost (From Table LXXXII) 7 per cent of $2,200,000	154,000	
Cost of producing energy from the alternative steam plant		309,000
Operation (From Table LXXXVI)	197,000	
Interest Charges (From Table LXXXVI) 7 per cent of $1,600,000	112,000	
Saving in cost of producing energy		52,500
Saving capitalized at 10.5 per cent equals value of lands and water rights		$500,000

To show the fallacy of this method, it is only necessary to point out that, as shown in Table LXXXII, a purchase price of $300,000 corresponds to a rate of return of 10.5 per cent on the total investment, and, if $500,000 were paid, the project would not earn 10.5 per cent and hence would not be as attractive as the steam plant, which does.

The error in this case is not as large as it would be if the value of the lands and water rights were a larger percentage of the total cost.

The correct method of stating the competitive plant method is as follows: *If the total price at which energy from a project must be sold in order to obtain a satisfactory rate of interest on the investment, exclusive of the purchase price of the lands, water rights, and existing structures, is less than the total price at which the same amount of energy from the alternative source must be sold to obtain the same rate of interest on the total investment; then the difference in the price at which energy must be sold, capitalized at the same rate of interest, is equivalent to the value of the lands, water rights, and existing structures.*

This correct rule would give the following value for the lands and water rights valued in Table LXXXII and is seen to be the same or $300,000.

TABLE LXXXVIII

ABBREVIATED FINANCIAL STATEMENT SHOWING VALUE OF LANDS AND WATER RIGHTS BY CORRECT COMPETITIVE PLANT METHOD

Price at which energy from the project must be sold		$333,500
To pay operating charges (Table LXXXII)	$102,500	
To pay 10.5 per cent interest on the construction cost of $2,200,000 (Table LXXXII)	231,000	
Price at which energy from the alternative steam plant must be sold to obtain the same per cent return (Table LXXXVI)		365,000
Difference		31,500
Difference, capitalized at 10.5 per cent equals value of lands and water rights		$300,000

A study of the foregoing discussion will show that the competitive plant method of valuation may be stated concisely as follows:

The per cent return on the total investment in the project, including the investment in lands and water rights, must be at least equal to the per cent return on the total investment in the best alternative source.

This brings us back to Criterion 2 of Sec. 465, which requires that the "satisfactory rate of return" used in the previous methods of valuation must be not only satisfactory, but at least equal to the rate obtainable from the best alternative source of power.

468. Subject Matter of Reports.—A report is essentially an opinion. However, the client has a right to expect that, unless otherwise specifically noted, all statements in the report are based on adequate and careful investigations by the engineer.

It is not always possible for the engineer to base his report entirely on information that has been verified by himself or his assistants. Therefore, when necessary assumptions must be made, they should be clearly stated to be such, and all unverified data should be carefully scrutinized to determine their probable accuracy. The report should contain a statement of the authority or the source of the unverified information and, if advisable, recommendations to the effect that verification of such information is necessary before the report can be accepted in its entirety.

The engineer should be careful not to pass upon the validity of contracts, franchises, and similar documents. The client's attention should be called to all assumptions that are of a legal nature or for any other reason are outside the province of the engineer, so that they can be verified by proper authorities.

The usual qualifications in the report of unverified data are similar to the following: "I am informed by your superintendent that, . . ." or "According to the publications of the Weather Bureau . . . ."

The engineer should also be careful to indicate the probable accuracy of his conclusions as affected by the data available. It is impossible to make exact estimates of cost of development as they are affected by probable labor rates, probable floods, the amount of good weather during construction, and other influencing conditions which are impossible to predict. Consequently the engineer is compelled to base his conclusions to some extent on pure judgment. The experienced engineer will therefore form his own conclusions as to the values derived from poor, average, and good working conditions, even if only one value is given in his report. He will, however, always state in his report on which conditions the value is based.

Many reports are improperly qualified by the statement that "under normal and proper conditions, the conclusions will be substantiated." Other reports frequently state that "the estimates and conclusions are conservative," without giving the degree of conservatism. A large corporation, promoting a number of projects, may be content with statements of *probable* value based on *average* conditions, on the assumption that, if the conditions are worse on one project they will be better on another, and thus prove satisfactory as a whole.

On the other hand, a client investing in only one project will not be satisfied with a value based on average conditions. With bad luck, the project

may fall short of being a successful enterprise. In such cases the conclusions in the report should be based on conditions which are worse than the average. In many cases it may be advisable to give the client values based on two possible extremes as well as a probable value. This will allow him to take whatever gamble he desires, if the worst value is not entirely satisfactory.

For cases where, due to differences in runoff, the output of the development will be greater in some years than in others, the usual basis of the report is the result for the average year. It is advisable, however, to give also the result for a minimum year if the result for several lean years immediately after starting the plant might cause embarrassment to the company.

There is an inherent desire on the part of promoters to have the engineer base his report on as favorable assumptions as the available data will permit. On the other hand, the attitude of investors is one of conservatism. The ethics of the engineering profession require that the engineer's opinion, as expressed in the report, should be unbiased and should not depend upon whether the report is for a promoter or an investor. This is not a statement that the engineer's opinion should be an honest one; it is merely another way of expressing the need for proper qualification or explanation of such opinion as hereinbefore described.

No engineer is an expert in all branches of his profession. It is never an admission of incompetency for an engineer to request consultations with other engineers having experience along special lines. Reports of geologists, contractors, foresters, foundation experts, and others may properly be appended to the engineer's main report.

469. Engineers' Investigations and Studies.—As it is impossible to estimate precisely, all prognostications of value are subject to errors. It is desirable, however, that errors should be such as properly to protect the interests of the investor. By far the greatest number of unfortunate errors in estimates of cost are caused by insufficient investigations, mostly the result of the client's desire for the completion of the report in too short a period of time. In such cases, the conclusions of the engineer should be properly qualified with recommendations for additional investigations. Other frequent sources of unfortunate errors are insufficient subsurface investigations to determine the nature of required excavations for structures, borrow-pits, canals, and other needs, and to establish the elevation of good foundations; lack of proper appreciation of possible inaccuracies in estimates of water supply; too optimistic ideas of the value of the output; and the assumption of ideal construction and operating conditions.

The engineer should always investigate thoroughly the degree of accuracy of maps compiled from surveys of other engineers, preferably by consultation with the chief of the survey party and the engineer who plotted the map. Very frequently, neatly drawn maps, containing contours at frequent intervals, have been made from a hasty reconnaissance and are intended for preliminary purposes only.

A careful record of all field investigations should be made. These are usually considered the property of the client for use in later studies and for designing and construction purposes.

TABLE LXXXIX

A Form for the Compilation of Project Data

	Reference
1. Name	
2. File Number of Calculations	
3. Date of Calculations	
4. River	
5. Location	
6. Owner .... 7. Promoter	
8. Purpose of Calculations and Accuracy	
9. Description of Development and Remarks	
10. Runoff Data:	
11. Drainage Area .... Sq. Mi.	
12. Per Cent of Flow Owned by (6) .... %	
13. Assumed Storage .... Billion Cu. Ft. Regulated for	
14. Location of Storage and Reference to Report	
15. Ownership of Storage	
16. Reference to Duration Curve of Runoff	
17. Tabulation of Runoff as shown by Duration Curve:	

	1 Designation	2 Per Cent of Time	3 Sec.-Ft. Flow	4 Sec.-Ft. Years	Reference
18.	Primary	100%			
19.	Secondary	58%			
20.	Pri. plus Sec.	58%			
21.	Dump	____%			
22.	Total				

	Reference
23. Floods	
24. 50-Year Flood .... Sec.-Ft. W. S. at Dam El.	
25. 100-Year Flood .... Sec.-Ft. W. S. at Dam El.	
26. 1000-Year Flood .... Sec.-Ft. W. S. at Dam El.	
27. Pondage Data:	
28. Area of Pond at Permanent Crest of Dam (47) .... Acres	
29. Usual Maximum Drawdown of Pond below (47 or 48) .... Ft.	
30. Capacity of Pond at (29) .... Acre Ft.	
31. Capacity of Pond at (29) in flow-days	
32. Primary flow-days	
33. Primary plus Secondary flow-days	
34. Mean-Drawdown Corresponding to (29) below (47 or 48) .... Ft.	
35. Possible Maximum Drawdown of Pond below (47 or 48) .... Ft.	
36. Capacity of Pond at (35) .... Acre Ft.	
37. Effect on Pondage of the Operation of Upper Plants	
38. Load Factor and Load Curve:	
39. Reference to Pondage Limitations	
40. Adopted Load Factor for Primary plus Secondary Power	
41. At Shaft .... %	
42. At Switchboard .... %	
43. For Delivered Power .... %	
44. Elevations:	
45. Datum of Elevations	
46. Conversion Factor to get U.S.G.S. Datum .... Ft.	
47. Permanent Crest of Dam .... El.	
48. Top of Temporary Flashboards .... El.	
49. Top of Concrete Non-Overflow Dam .... El.	
50. Top of Embankment at Dam .... El.	
51. Average Tail-water at Next Plant Above .... El.	
52. Average Head-water Surface .... El.	
53. Average Tail-water Surface .... El.	
54. High Tail-water Surface for .... Year Flood (23) .... El.	
55. Minimum Tail-water Surface .... El.	
56. Head:	
57. Nominal Productive Head (Exact Information Lacking) .... Ft.	
58. Gross Head with Full Pond .... Ft.	
59. Average Gross Head .... Ft.	
60. Productive Head .... Ft.	
61. Minimum Net Head at Peak Demand Capacity (73) .... Ft.	
62. Maximum Net Head at Full Load of One Unit (76) .... Ft.	
63. Maximum Net Head at Full Load of Plant (77) .... Ft.	

TABLE LXXXIX—*Continued*

		Col. 1 Average Load	Col. 2 Peak Load	Reference
64.	Installation:			
65.	Assumed Turbine Efficiency	%	%	
66.	Assumed Generator Efficiency	%	%	
67.	Assumed Transformer Efficiency	%	%	
68.	Assumed Transmission Efficiency	%	%	
69.	Assumed Transformer Efficiency	%	%	
70.	Assumed Other Equipment Efficiency	%	%	
71.	Assumed Total Efficiency	%	%	
72.	Average Output of Primary plus Secondary from (60) and Col. 3 of (20)		Shaft HP	
73.	Peak Demand Capacity of Primary plus Secondary from (72) and (40). The Turbines Must Have This Capacity at (61)		Shaft HP	
74.				
75.				
76.	Maximum Capacity of Each of .... Units at (62) (Maximum Load into Generator)		Shaft HP	
77.	Maximum Capacity of Plant at (63)		Shaft HP	
78.	Maximum Discharge of Plant at (63)		Sec. Ft.	
79.	Kw. Rated Capacity of Each of .... Generators		Kw.	
80.	K.-va. Rated Capacity of Each of .... Generators at ...... % Power Factor		K.-va	
81.	Output in Kw.-hr. per Annum:	Switchboard	Delivered	
82.	Primary			
83.	Secondary			
84.	Total Primary plus Secondary			
85.	Dump			
86.	Total			

		Alternatives 1	Alternatives 2	Alternatives 3	Reference
87.	Cost of Development:				
88.	Lands and Water Rights for Sale	$	$	$	
89.	Addit'l Lands and Water Rights Required	$	$	$	
90.	Construct'n Cost Exclusive of Transmission	$	$	$	
91.	Cost of Transmission to ______	$	$	$	
92.	Other Costs	$	$	$	
93.	Total Cost of Development	$	$	$	
94.	Total Cost per h.p. of Peak Demand (73)	$	$	$	
95.	Annual Operating Charges:				
96.	Operators	$	$	$	
97.	Power House $				
98.	Outside Men $				
99.	Transmission $				
100.	Substations $				
101.	Miscellaneous $				
102.	Maintenance and Repairs	$	$	$	
103.	Power Development $				
104.	Transmission $				
105.	Substations $				
106.	Miscellaneous $				
107.	Taxes	$	$	$	
108.	Insurance	$	$	$	
109.	Depreciation	$	$	$	
110.	Storage Charge	$	$	$	
111.	Management	$	$	$	
112.	Miscellaneous Charges	$	$	$	
113.	Credit Miscellaneous Income	$	$	$	
114.	Total Annual Charges	$	$	$	
115.	Interest Charges at ...% of (93)	$	$	$	
116.	Total Cost of Producing Power (114) + (115)	$	$	$	
117.	Cost of Producing Power, mills per kw.-hr. from (118) and (81)				
118.	Where Energy is Measured				
119.	Primary Output (82)				
120.	Secondary Output (83)				
121.	Total Primary Plus Secondary (84)				
122.	Dump Output (85)				
123.	Total Primary, Secondary and Dump (86)				
124.	General Notes				

The engineer's detailed office studies and calculations generally remain in his possession. The need in these for careful and complete compilation of all assumptions, data, and calculations, properly indexed, logically arranged, and cross-referenced, cannot be overemphasized. The basic data in all sec-

tions of the calculations should give the source from which they were obtained or a reference thereto, and all studies should be explained to an extent sufficient to allow another engineer to follow them. The engineer should bear in mind the possibility that, after several years, he may be called upon to expand or explain parts of the report and perhaps to testify in a legal action in connection with the subject of the report.

The writer has found the form given in Table LXXXIX useful for compiling a summary of data for quick reference. The form is particularly well adapted to the compilation of data for the various projects of a large system.

Table XC gives an outline of the essential data required for a report on a plant or a project. A list of this kind is useful as a guard against omissions in field investigations and in the compilation of the report.

TABLE XC

COMBINED OUTLINE OF ESSENTIAL DATA REQUIRED FOR A REPORT ON A PLANT OR A PROJECT

I. Scope of Report:
  1. Client,
  2. Purpose of report,
  3. Type of report desired.

II. Corporate History of Existing Company:
  1. General information,
  2. Officers,
  3. Directors,
  4. Controlling interests.

III Charters, Franchises, and other Assets:
  1. Charters,
    (a) Date, life, and powers,
    (b) Amendments.
  2. Franchises,
    (a) Date and life,
    (b) Rights secured and pending,
    (c) Conditions and restrictions,
    (d) Competitive franchises, with particulars.
  3. Other incorporeal assets,
    (a) Contracts,
      (1) Date and life,
      (2) Between what parties,
      (3) Conditions,
    (b) Leases and mortgages,
      (1) Date and life,
      (2) Between what parties,
      (3) Conditions.
  4. What charters and franchises are required for proposed new work and extensions?

IV. Physical Features:
  1. Location and name of river,
  2. Nearest railroad point,
  3. Transportation facilities to site, existing and proposed,
  4. Drainage area,
  5. Topographical conditions of the watershed,
  6. Geological conditions of the watershed,
  7. Climatic conditions,
    (a) Range in temperature,
    (b) Variations in precipitation by seasons.

TABLE XC—*Continued*

8. Existing storage, controlled and not controlled,
9. Water supply,
   (*a*) Available stream gagings,
   (*b*) Precipitation records,
   (*c*) Average precipitation,
   (*d*) Estimated present stream flow,
   (*e*) Estimated stream flow after proposed storage,
   (*f*) Estimated flood characteristics,
   (*g*) Effect of plants above on variations in stream flow,
   (*h*) Requirements of plants below, restricting variations in stream flow,
   (*i*) Floating substances in the water,
      (1) Ice, surface, frazil, and anchor,
      (2) Effect of proposed new ponds on ice,
      (3) Silt,
      (4) Débris,
      (5) Acids and other impurities,
   (*j*) Condition of dams above if failure will cause damage.

V. Elevations:
1. Datum of elevations,
2. Conversion factor to get U. S. G. S. datum,
3. Permanent crest of spillway, existing or proposed,
4. Top of temporary flash-boards,
5. Top of concrete non-overflow dam,
6. Top of dam embankment,
7. Average tail-water of next plant above, if affected,
8. Average tail-water surface,
9. Highest tail-water surface,
10. Lowest tail-water surface.

VI. Head:
1. Gross head,
2. Productive head (see definition, Sections 49 and 50),
3. Minimum net head,
4. Maximum net head,
5. Rating curve of tail race,
6. Effect of possible ice gorges on tail-race elevation.

VII. Maps, Profiles, Photos, Drawings and Literature:
1. Maps and profiles,
   (*a*) General map of surrounding territory covering sites, neighboring cities, railroads, highways, elevation above sea level, etc., preferably a contour map,
   (*b*) Map showing nature and size of drainage basin, giving elevations above sea level, preferably a contour map,
   (*c*) Map of reservoir sites showing contours at various stages of water surface.
   (*d*) General map showing,
      (1) Lands owned, leased, or under option, and required for extension or new work, and location of present and proposed structures,
   (*e*) Contour map and sections at sites of all structures showing foundation conditions,
   (*f*) Profile of river,
   (*g*) Get if possible,
      (1) U. S. G. S. maps,
      (2) U. S. Coast and Geodetic Survey maps,
      (3) Other national state or municipal maps,
      (4) Maps of private surveys.
2. Photos: Get photos of present structures and sites of proposed structures,
3. Drawings: Get blue prints of all present and proposed structures,
4. Literature: Get copies of all available reports.

TABLE XC—*Continued*

VIII. Legal:
1. Has the Power Company the right under the local law to condemn,
   (*a*) The flow of the streams,
   (*b*) The necessary real estate?
2. How will water rights be acquired?
3. How will lands and rights-of-way be secured?
4. What approval of City, State or National Government is necessary for the proposed extension and new work?
5. What legislation is necessary?
6. What parts of items 4 and 5 have been secured?
7. How long will it take to secure such approval and legislation?
8. Are fishways required?
9. Will log chutes be required?
10. Can transmission-line right-of-way be secured on sectional lines or public highways free of charge?
11. What will taxes on proposed extensions and new work be?
12. What will the taxes on existing work, bonds and stock be?

IX. Power Available:
1. Kw.-hr. per annum of primary output,
2. Kw.-hr. per annum of secondary output in average year,
3. Kw.-hr. per annum of secondary output in minimum year,
4. Kw.-hr. per annum of dump output in average year,
5. Kw.-hr. per annum of dump output in minimum year,
6. Distribution of foregoing items during week and year,
7. Load factor required for delivered power.

X. Power Markets:
1. Communities being or to be served,
2. Existing and prospective power contracts,
3. Large power users,
4. Street lighting,
5. Domestic users,
6. Load curves,
7. Load factors,
8. Probable growth of market,
9. Probable sale price of power or existing rates,
10. Probable competition,
11. Probable cost of steam power for the market.

XI. Description of Existing Physical Property:
1. Lands,
   (*a*) Location,
   (*b*) Description,
   (*c*) Areas,
   (*d*) Detailed actual or estimated value,
   (*e*) Used for what purpose at present,
   (*f*) Owned in fee,
   (*g*) Leased (description of lease).
2. Water rights,
   (*a*) Location,
   (*b*) Nature,
   (*c*) Detailed actual or estimated value,
   (*d*) Owned,
   (*e*) Leased (description of lease),
   (*f*) Controlled by contract (description of contract).
3. Structures and Apparatus:
   (*a*) Itemized list (see Table XCI),
   (*b*) Description, general dimensions, capacities, etc.,
   (*c*) Drawings,
   (*d*) Character of foundations,
   (*e*) Age, condition and probable life,
   (*f*) Detailed actual or estimated cost,

TABLE XC—*Continued*

- (*g*) Provisions for extensions,
- (*h*) Owned,
- (*i*) Leased (description of lease

4. Reservoirs and ponds,
   - (*a*) Areas,
   - (*b*) Permissible draft,
   - (*c*) Capacity,
   - (*d*) High water,
   - (*e*) Relocated roads, bridges, railroads, etc.

XII. Proposed New Work and Extensions:

1. Lands now owned or leased (see XI–1),
2. New lands required,
   - (*a*) When needed,
   - (*b*) Area,
   - (*c*) Location,
   - (*d*) Required for what purpose,
   - (*e*) Used for what purpose at present,
   - (*f*) Under option (description of option),
   - (*g*) Otherwise controlled,
   - (*h*) Not controlled,
   - (*i*) Detailed probable cost.
3. Water rights, now owned or under contract (see XI–2),
4. Additional water rights required,
   - (*a*) Location and when needed,
   - (*b*) Nature,
   - (*c*) Under option (description of option),
   - (*d*) Otherwise controlled,
   - (*e*) Not controlled,
   - (*f*) Detailed probable cost,
5. Proposed new construction,
   - (*a*) When needed,
   - (*b*) Itemized list (see Table XCI),
   - (*c*) Description, general dimensions, capacities, etc.,
   - (*d*) Sketches or drawings,
   - (*e*) Description of subsurface investigations,
   - (*f*) Character of foundations,
   - (*g*) Detailed estimate of construction cost,
6. Reservoirs and ponds to be created,
   - (*a*) Areas,
   - (*b*) Permissible draft,
   - (*c*) Capacity.

XIII. Data for Estimates of Cost:

1. Kind and location of material for construction,
   - (*a*) Ingredients for concrete,
   - (*b*) Stone for plums and rip-rap,
   - (*c*) Earth for embankments,
   - (*d*) Clay for puddle,
   - (*e*) Timber lands,
   - (*f*) Are foregoing on owned or controlled lands or lands to be purchased?
2. Local costs,
   - (*a*) Skilled labor,
   - (*b*) Unskilled labor,
   - (*c*) Teams,
   - (*d*) Trucks,
   - (*e*) Coal,
   - (*f*) Lumber,
   - (*g*) Other materials.
3. Available camp sites,
4. Freight rates,
5. Local hauling rates,
6. Local rates for power for construction purposes.

TABLE XC—*Continued*

XIV. Operation of Existing Properties:
1. Statement of assets and liabilities for a period of five years,
   (*a*) General balance sheet,
   (*b*) Nature of funded debt,
      (1) Interest rates and dates,
      (2) Date of maturity of bonds,
      (3) Premium and date at which bonds may be redeemed,
   (*c*) Nature of floating debt,
   (*d*) Contingent liabilities (i.e., Cumulative dividends, etc.),
   (*e*) Debts not due by ascertained amounts (i.e., taxes, sinking fund, depreciation, suits pending, adverse legislation, etc.),
   (*f*) Profit and loss account,
   (*g*) Schedule of construction costs.
2. Statements of Receipts (for period of five years),
   (*a*) Lighting,
      (1) Commercial,
      (2) Municipal,
   (*b*) Power,
      (1) Commercial,
      (2) Municipal,
   (*c*) Other sources.
3. Statements of Operating Expenses (for a period of five years):
   (*a*) Manufacturing,
      (1) Fuel, oil, waste and supplies,
      (2) Labor,
      (3) Maintenance,
   (*b*) Distribution,
      (1) Labor,
      (2) Maintenance,
   (*c*) General,
      (1) Office salaries,
      (2) Supplies,
      (3) Rents,
      (4) Taxes,
      (5) Legal,
      (6) Insurance,
      (7) Damages.
4. Statement of Net Earnings (for period of five years),
5. Statement of Deductions (for a period of five years),
   (*a*) Fixed charges,
   (*b*) All other.
6. Statement of the Surplus (for a period of five years),
7. Statement of Special Cause for Material Increase or Decrease in Earnings and Operating Expenses During any Year,
8. Statistics (for a period of five years),
   (*a*) Power-station load curves showing peaks and load factor,
   (*b*) Cost per kw.-hr. generated (i.e., switchboard cost),
   (*c*) Cost of power per kw.-hr., if purchased, and contract conditions,
   (*d*) Gross receipts per kw.-hr.,
   (*e*) Gross receipts per capita served,
   (*f*) Per cent of operating expense to gross receipts,
   (*g*) Water consumption and cost,
   (*h*) Kw.-hr. generated per annum,
   (*i*) Output of substations,
   (*j*) Kind of fuel, consumption and cost delivered.
9. Fund for Depreciation.

XV. Possible Competition:
1. By existing competitors,
2. By possible future competitors,

470. Investigations for Market for Power.—The investigation of the market for power, upon which the estimated income is based, involves the determination of the following general items:

1. The amount of demand;
2. The type of industries to be served;
3. The load curve and load factor;
4. The power factor;
5. The required voltage and frequency;
6. The probable future growth;
7. The probable sale price.

The amount of power demanded must, of course, be equal to the possible output of the proposed development.

The type of industries to be served affects the required reliability of service, voltage regulation, and other features.

The load curve will show the distribution of the demand during the year, the week, and the day. (See discussion in Sec. 40.) Its shape determines the load factor and fixes the peak capacity of the required installation and the size of the pond necessary to regulate the flow properly.

Information as to the probable power factor, and the required voltage and frequency is an obvious necessity.

The probable growth of the market is of interest if it is desired to preserve a monopoly in the service of the district. For this, a sufficient number of projects must be held in reserve.

The gross income is directly proportional to the sale price of the different classes of power.

The best type of market from an investor's standpoint, and the easiest to analyze, is a contract with a neighboring system to furnish a given amount of energy at a certain load factor. Here the essential features of the new load are described in the contract or easily obtained.

In the case of projects to supply the normal market growth of a large system, the characteristics of the new market may be considered about the same as that of the existing market and, if slightly different, the percentage of error will be small in comparison with the whole load. The probable growth is usually obtained by projecting a curve showing demands in preceding years.

A close determination of the probable characteristics of the market requirements of a district not previously served by any form of central station is very difficult unless by far the greater part of the load is to be taken by a few large manufacturers who have good data on their demands. If the general nature of the market is to be mostly domestic service, street lighting, and a number of small power users, the determination is usually made by a comparison with the load requirements of similar communities already served.

The peak capacity of the project to serve a given market is always less than the sum of the peak demands of the individual users, because of the diversity of the loads and the fact that the individual peak demands do not occur simultaneously. The ratio of the sum of the individual demands to the combined demand is the "diversity-factor" and may be as high as 3 for

residence lighting, 2 for commercial lighting and 1.5 for a market supplying a large number of small diversified customers.[5]

It is possible, of course, to make a canvass of all probable individual consumers; but, when all the necessary data have been gathered, the probable error in the determination of the combined load factor and the combined peak demand for a greatly diversified system is greater than that from a comparison with a similar district already served.

It is usual, however, to determine approximately the proportion of residence lighting, commercial lighting, small power users, etc., for use in making an intelligent comparison with a similar community in which the proportion is not the same.

If the price for energy can be made attractive, an independent system or a large factory may be induced to shut down its steam plant entirely. The energy from the project must then compete with the bare cost of steam power, excluding all interest and other fixed charges which would not be eliminated by the change. Another arrangement would be to sell only low-priced secondary or surplus energy to the owner of the steam plant, allowing the steam plant to run during periods of low water.

471. Estimates of Cost.—The tendency is to estimate too low on all types of projects, and estimates of water powers are no exception. The greatest source of errors in estimating is the failure to provide sufficient time for adequate investigations and studies. Other frequent causes of errors are mentioned in Sec. 469.

Table XCI contains a list of most of the important items which enter into the cost of a hydro-electric development. Such a list should be kept by engineers and consulted after each estimate to guard against omissions.

Manufacturers of turbines, generators, and other apparatus are always willing to give accurate estimates of cost, and such estimates are usually very reliable if sufficient time is given for their preparation. Bids are frequently obtained from prospective contractors on the general construction items. Estimates based on these data, supplemented by the engineer's judgment, are the most acceptable to the prospective investor.

An estimate of cost is practically worthless unless based on first-hand knowledge of the estimator. An able engineer will not attempt to estimate the cost of an important item upon which he has had no experience, without consulting someone who has. The use of published or off-hand statements of unit costs on other jobs is to be emphatically discouraged. They seldom are reliable unless accompanied by an ample description of the costs included therein, as they are sometimes bare labor and material costs excluding plant, miscellaneous, and other overhead expenses which are a large percentage of the total. Moreover, they frequently apply to conditions which are far different from those pertaining to the project. Published unit-price bids on other jobs are likely to be unbalanced.

[5] For a discussion of diversity-factor, see Sec. 40 and also "Diversity-factor," by H. B. Gear, Elec. World, Vol. LV, p. 927 and "Electric Central Station Distribution Systems," by Gear and Williams, D. Van Nostrand Co., 1911. Also Commercial Bulletin and Data on Consumers' Demand and Load Factor, National Elec. Lt. Assn.

An allowance for unforeseen contingencies, omissions, miscellaneous items not large enough to be estimated separately, and possible errors in both estimated costs and quantities is usually provided at or near the end of the estimate. (See Item XXVII of Table XCI.) The amount depends upon the extent of the data available and the probable accuracy of the investigations and studies. It varies between 5 and 20 per cent of the total estimated cost.

Interest during construction is approximately the interest on the total estimated cost for about one-half the construction period; but, for projects taking several years to build, it is advisable to estimate a progress schedule and figure the interest charges more accurately.

Working capital is included only if the returns from power sales will not be quick enough to provide money for running expenses when the plant first starts up. This is probablv necessary only for a new company.

TABLE XCI

ESTIMATING FORM, HYDRO-ELECTRIC POWER DEVELOPMENTS

I. Preliminary Expenses:
- (a) Surveys,
- (b) Reports,
- (c) Borings and test pits,
- (d) Miscellaneous,
- (e) Water supply and sanitation,
- (f) Office and camps (see also XXII),
- (g) Power-transmission line for construction purposes.

II. Clearing Site:
- (a) Removing structures and apparatus,
- (b) Removing trees and brush,
- (c) Grubbing,
- (d) Salvage.

III. Highways and Bridges:
- (a) Relocation of existing,
- (b) For access to site,
- (c) For local construction purposes,
- (d) Local permanent roads and bridges,
- (e) Special structures.

IV. Railroads and Bridges:
- (a) Relocation of existing,
- (b) For access to site,
- (c) For local construction purposes (except as included in contractor's plant)

V. Wharves:
- (a) Relocation of existing,
- (b) For access to site,
- (c) For local construction purposes.

VI. Ferries:
- (a) Relocation of existing,
- (b) For access to site,
- (c) For local construction purposes.

VII. Taking care of Water:
- (a) Earth excavation,
- (b) Rock excavation,
- (c) Embankment,
- (d) Cofferdam,
- (e) Sheet piling,

TABLE XCI—*Continued*

(*f*) Round piles,
(*g*) Heel fill,
(*h*) Sand-bag fill,
(*i*) Pumping,
(*j*) Gates and sluices,
(*k*) By-pass conduit (see XI),
(*l*) Removal of cofferdam, etc.

VIII. Reservoirs and Pond:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Drift barriers.

IX. Dams:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Ice and trash fenders,
(*i*) Fishway,
(*j*) Log, ice and trash chute,
(*k*) Sluice gates,
  (1) Gates,
  (2) Hoists,
  (3) Sluice lining,
  (4) Screens,
  (5) Electrical equipment,
(*l*) Bridge on crest,
(*m*) Movable crest and operating mechanism,
(*ma*) Air supply for (m) to prevent freezing,
(*n*) Lighting,
(*o*) Intake and equipment (see X),
(*p*) Masonry dams,
  (1) See IX(*a*) to IX(*o*), inclusive,
  (2) Earth excavation (see also IX(*p*)12*a*),
  (3) Rock excavation (see also IX(*p*) 12*b*),
  (4) Preparing foundations and grouting (see also IX(*p*) 12*e*),
  (5) Bearing piles,
  (6) Masonry (see also IX(*p*) 12*d*),
  (7) Backfill,
  (8) Under drainage,
  (9) Steel reinforcement,
  (10) Miscellaneous steelwork,
  (11) Apron,
  (12) Cut-off,
    a. Earth excavation,
    b. Rock excavation,
    c. Sheet piling,
    d. Masonry,
    e. Grouting (see also IX (*p*) 4),
  (13) Water-stops,
  (14) Heel fill,
  (15) Expansion joints.
(*q*) Timber dams:
  (1) See IX(*a*) to IX(*o*) inclusive,
  (2) Earth excavation,
  (3) Rock excavation,

TABLE XCI—*Continued*

(4) Concrete footings,
(5) Bearing piles,
(6) Sheet piling,
(7) Grouting,
(8) Timbers,
(9) Sheeting,
(10) Stone filling,
(11) Earth filling,
(12) Miscellaneous steel, bolts and pins,
(13) Heel fill,
(14) Apron.

(*r*) Embankment:
(1) See IX(*a*) to IX(*o*) inclusive,
(2) Earth excavation (see also IX(*r*) 7*a*),
(3) Rock excavation (see also IX(*r*) 7*b*),
(4) Earth fill,
(5) Rock fill,
(6) Retaining walls,
(7) Core-walls and cut-offs,
a. Earth excavation,
b. Rock excavation,
c. Masonry,
d. Steel reinforcement,
e. Water-stops,
f. Puddle,
g. Sheet piling,
h. Grouting,
(8) Slope protection,
(9) Sodding and seeding,
(10) Gutters and drains,
(11) Roadway surfacing,
(12) Outlet tower (see X),
(13) Bridge to tower (see X),
(14) Outlet under embankment (see XI).

X. Intakes:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Ice and trash fender and chutes,
(*i*) Bridge on top,
(*j*) Lighting,
(*k*) Structure,
(1) Earth excavation (see X(*k*) 10*a*),
(2) Rock excavation (see X(*k*) 10*b*),
(3) Preparing foundations and grouting (see also X(*k*) 10*e*),
(4) Bearing piles,
(5) Masonry (see also X(*k*) 10*d* and X(*k*) 14),
(6) Backfill,
(7) Under drainage,
(8) Steel reinforcement,
(9) Miscellaneous steelwork,
(10) Cut-off,
a. Earth excavation,
b. Rock excavation,
c. Sheet piling,
d. Masonry,
e. Grouting (see also X(*k*) 3),
(11) Water-stops,

TABLE XCI—*Continued*

(12) Air inlets,
(13) Superstructure (see XIII),
a. Housing over air inlet
b. Housing over gates,
(14) Heating,
(15) Retaining walls (see also X(*k*) 5).

(*l*) Equipment,
(1) Racks and supports,
(2) Main gates and supports,
(3) Main gate hoists,
(4) Exciter gates and supports,
(5) Exciter gate hoists,
(6) Filler gates and supports,
(7) Filler gate hoists,
(8) Electrical operation with cables and ducts,
(9) Miscellaneous steel,
(10) Stop-logs and supports,
(11) Miscellaneous equipment,
(12) Mechanical rakes,
(13) Cranes,
(14) Air supply to prevent freezing.

XI. Conduits:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Intakes (see X),
(*i*) Culverts,
(*j*) Sewers,
(*k*) Cattle crossings,
(*l*) Settling basins,
(*m*) Waste weirs (see IX),
(*n*) Waste channels,
(*o*) Bridges and trestles for conduits,
(*p*) Fencing,
(*q*) Canals,
(1) See XI(*a*) to XI(*p*) inclusive,
(2) Earth excavation,
(3) Rock excavation,
(4) Embankment (see IX(*r*)),
(5) Lining and slope protection,
(6) Cable crossings,
(7) Drops,
(8) Reinforcing steel,
(9) Retaining walls,
(10) Backfill,
(11) Outside walls (see IX),
(12) Aqueducts (see XI(*t*),
(13) Emergency gates.

(*r*) Pipe lines,
(1) See XI(*a*) to XI(*p*), inclusive,
(2) Earth excavation (for bench),
(for piers),
(3) Rock excavation,
(4) Embankment (see IX(*r*)),
(5) Covering and refill,
(6) Pipe materials,
(7) Circumferential stiffness,
(8) Cradles,

TABLE XCI—*Continued*

(9) Piers,
(10) Anchorages,
(11) Reinforcing steel,
(12) Painting,
(13) Expansion joints,
(14) Air inlets,
(15) Air outlets,
(16) Blow-offs,
(17) Drainage of pipe,
(18) Under drainage,
(19) Valves,
(20) Relief valves,
(21) Man-holes,
(22) Special connections,
(23) Surge tank (see XI(*v*)),

(*s*) Penstocks (see XI(*r*)),

(*t*) Flumes,
(1) See XI(*a*) to XI(*p*) inclusive,
(2) Earth excavation,
(3) Rock excavation,
(4) Embankment (see IX(*r*)),
(5) Flume materials,
(6) Flume footings,
(7) Foundation drainage,
(8) Drops,
(9) Reinforcing steel,
(10) Painting,
(11) Expansion joints,
(12) Emergency gates,

(*u*) Tunnels,
(1) See XI(*a*) to XI(*p*) inclusive,
(2) Portals,
a. Earth excavation,
b. Rock excavation,
c. Refill,
d. Bank protection,
e. Paving,
f. Masonry,
g. Steel reinforcement,
h. Drains,
(3) Tunnel earth excavation,
(4) Tunnel rock excavation,
(5) Bracing,
(6) Masonry lining,
(7) Crown fill,
(8) Invert fill,
(9) Cut-offs around lining,
(10) Lining drains,
(11) Invert drains,
(12) Grouting,
(13) Reinforcing steel,
(14) Air inlets,
(15) Air outlets,
(16) Blow-offs,
(17) Special connections,
(18) Valves,

(*v*) Surge tanks,
(1) See XI(*a*) to XI(*p*) inclusive,
(2) Earth excavation,
(3) Rock excavation,
(4) Refill and grading,
(5) Foundation drains,
(6) Tank drains,

TABLE XCI—*Continued*

(7) Concrete footings,
(8) Reinforcing steel,
(9) Surge tank and tower,
(10) Frost-proofing,
(11) Painting,
(12) Heating apparatus,

XII. Power House Substructure:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Earth excavation,
(*i*) Rock excavation,
(*j*) Preparing foundations and grouting,
(*k*) Bearing piles,
(*l*) Under drainage system,
(*m*) Cut-off (see IX(*p*) 12),
(*n*) Refill and embankment,
(*o*) Masonry,
(*p*) Steel reinforcement,
(*q*) Miscellaneous steelwork,
(*r*) Power house drains and valves,
(*s*) Intake equipment (see X(*l*)),
(*t*) Air inlets,
(*u*) Ice and trash fenders,
(*v*) Water-stops,
(*w*) Water supply,
(*x*) Draft-tube forms,
(*y*) Stop-logs,
(*z*) Stop-log supports.

XIII. Power House Superstructure:
(*a*) Structural steel,
(*b*) Miscellaneous steel,
(*c*) Walls and partitions,
(*d*) Floors,
(*e*) Floor surfacing and hardeners,
(*f*) Floor covering,
(*g*) Sills and lintels,
(*h*) Roof,
(*i*) Roof covering,
(*j*) Gutters, leaders and flashing,
(*k*) Steel reinforcement,
(*l*) Doors, windows and skylights,
(*m*) Hardware,
(*n*) Painting and finishing
(*o*) Ventilators,
(*p*) Plumbing and water,
(*q*) Lighting,
(*r*) Heating,
(*s*) Bus structure (if not in XVI),
(*t*) Switch cells,
(*u*) Fire protection,
(*v*) Office equipment,
(*w*) Cornices and supports.

XIV. Hydraulic Equipment:
(*a*) Main turbines,
(*b*) Exciter turbines,

TABLE XCI—*Continued*

(*c*) Main governors, pumps, tanks and piping,
(*d*) Exciter governors, pumps, tanks and piping,
(*e*) Governor oil,
(*f*) Switchboard control,
(*g*) Hand control,
(*h*) Relief valves (see XI(*r*) 20),
(*i*) Water-cooling system for bearings,
(*j*) Oiling system.

XV. Electrical Equipment:
(*a*) Main generators,
(*b*) Exciter generators,
(*c*) Low-tension switches,
(*d*) Low-tension bus-bars.
(*e*) Switchboard,
(*f*) Wiring and ducts,
(*g*) Transformers,
(*h*) High-tension switches,
(*i*) High-tension bus-bars,
(*j*) Lightning arresters,
(*k*) Transformer oil,
(*l*) Oil filters and pumps,
(*m*) Oiling system,
(*n*) Oil cooling system,
(*o*) Motor generator sets,
(*p*) Storage batteries.

XVI. Miscellaneous Equipment:
(*a*) Crane and motor,
(*b*) Pumps and motors,
(*c*) Eductors,
(*d*) Machine shop,
(*e*) Transformer truck and track,
(*f*) Air compressor,
(*g*) Blowers.

XVII. Testing and Starting.

XVIII. Tail Race:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Earth excavation,
(*i*) Rock excavation,
(*j*) Embankment and refill,
(*k*) Paving,
(*l*) Sheet piling,
(*m*) Slope protection,
(*n*) Retaining walls,
(*o*) River walls and crib.

XIX. Outdoor Transformer Station Structure:
(*a*) Preliminary (see I),
(*b*) Clearing site (see II),
(*c*) Highways and bridges (see III),
(*d*) Railroads and bridges (see IV),
(*e*) Wharves (see V),
(*f*) Ferries (see VI),
(*g*) Taking care of water (see VII),
(*h*) Earth excavation,
(*i*) Rock excavation,

TABLE XCI—*Continued*

(*j*) Embankment and refill
(*k*) Slope protection,
(*l*) Retaining walls,
(*m*) Concrete footings,
(*n*) Concrete floor paving,
(*o*) Superstructure (see also XIII)
(*p*) Fence,
(*q*) Transformer track (see also XVI(*e*)),
(*r*) Drains,
(*s*) Structural steel,
(*t*) Equipment (see XV).

XX. Indicating and Recording Devices for Hydraulic Purposes:
(*a*) Venturi meters,
(*b*) Pitot tubes,
(*c*) Measuring weirs (see IX),
(*d*) Water-stage indicators,
(*e*) Water-stage alarms,
(*f*) Miscellaneous metering devices.

XXI. Transmission Line and Telephone Line:
(*a*) Foundations,
(*b*) Anchors,
(*c*) Steel towers,
(*d*) Poles,
(*e*) Cross arms,
(*f*) Insulators,
(*g*) Conductors,
(*h*) Ground wire and grounds,
(*i*) Stay guys and anchors,
(*j*) Telephone line and insulators,
(*k*) Transformers and meters,
(*l*) Special crossings,
(*m*) Sectionalizing stations.

XXII. Substations (see XV and XIX).

XXIII. Permanent Quarters:
(*a*) Operators' houses,
(*b*) Guest house,
(*c*) Garage and stables,
(*d*) Water supply and sanitation (see also I(*e*)).

XXIV. Construction Overhead (unless included in unit costs):
(*a*) Freight, haulage, erection and removal of construction plant,
(*b*) Organization and overhead expenses,
(*c*) Storeroom salaries and expenses,
(*d*) Watching, lighting and guarding,
(*e*) Provisions for safety to persons and property,
(*f*) Accidents and damage,
(*g*) Plant investment less salvage, or plant rental, or plant depreciation,
(*h*) Plant operation and maintenance and repairs,
(*i*) Small tools,
(*j*) Stable, garage, commissary, camp and other auxiliary, feature investment and operation,
(*k*) Fire, payroll and personal injury insurance,
(*l*) Contractor's bond.

XXV. Local General Charges:
(*a*) Engineering and office equipment,
(*b*) Engineering salaries and expenses,
(*c*) Engineering supplies,
(*d*) Office salaries and expenses,
(*e*) Fire, payroll and personal injury insurance,
(*f*) Engineers' quarters,
(*g*) Auxiliary features (see XXIV(*j*)).

TABLE XCI—(*Continued*)

XXVI. **Engineering and Miscellaneous:**
- (*a*) Home office engineering, supervision, purchasing and inspection,
- (*b*) Consulting engineering,
- (*c*) Legal expenses,
- (*d*) Organization expenses,
- (*e*) Client's office salaries and expenses,
- (*f*) Entertaining,
- (*g*) Royalties, franchises and licenses.

XXVII. Allowance for Unforeseen Contingencies and Omissions.

XXVIII. Real Estate, Rights of Way, Flowage Rights and Water Rights.

XXIX. Interest during Construction.

XXX. Taxes during Construction.

XXXI. Interest and Taxes on Cost of Real Estate, Rights-of-way, Flowage Rights and Water Rights during Period of Idleness Prior to Construction.

XXXII. Working Capital.

472. Estimates of Annual Charges.—The annual charges are usually confined to the items listed in Table LXXXI although, of course, there may be other items required for special cases. A careful tabulation, by the writer, of the operating charges of a number of systems discloses a wide variation of results, depending upon the character of the installations, the policy of the operating managers, the systems of operation, the methods of classification of accounts, and the number of plants in the system to absorb the overhead expenses. Fortunately, in most cases, the operating charges are small in comparison with depreciation reserves and interest charges, and errors in estimating are not a large percentage of the total.

The recommendations given hereinafter are intended to be only rough approximations which must be supplemented by the experience and judgment of the engineer.

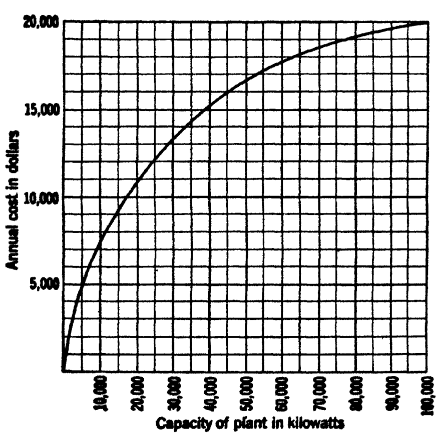


FIG. 478.—Approximate Annual Cost of Inside Operators of Hydro-electric Developments.

(*a*) *Power Development Operators.*—The operators of a power development may be divided roughly into two classes, i.e., inside and outside men.

The cost of inside men varies not only with the size of the installation and the number of units, but is much less for automatic and remotely operated plants than for those that are not so equipped.

Figure 478 shows the approximate cost of inside men for three or four unit power houses which are not automatically or remotely operated.

The costs in Fig. 478 include all oil, waste and ordinary supplies, but only the salaries of men required for operating the units, keeping them properly

cleaned, adjusted, and oiled, and making such miscellaneous small repairs as are required in the course of ordinary operation. The cost of extraordinary repairs, including the salaries of a permanent repair gang, if the plant is large enough to require one, is listed under " maintenance and repairs."

It is generally assumed that automatic and remotely operated plants require only occasional inspection and that the cost of inside operators is negligible. Very small plants have been operated on this basis; but large, important installations should always have one or two men on hand for emergencies. Plants so equipped have only recently come into use and, until more data have become available, it is best to assume that the cost of inside operators for plants of this type is from one-quarter to one-third that of plants not so equipped.

The cost of outside men includes the labor required for raking racks, installing flash-boards, taking care of ice and trash, operating gates, and similar work, together with such miscellaneous small repairs and adjustments as are required in the course of ordinary routine. It is impossible to give even approximate data on such costs on account of the widely divergent character of different installations. The cost may vary from zero for the best conditions and where the duties of the inside operators will permit of a small amount of necessary outside work, to as much as the cost of inside operators for extreme conditions.

(*b*) *Transmission Patrol.*—The cost of transmission-line patrol varies from about \$50 to \$100 per mile per annum, depending upon the nature of the country.

(*c*) *Substation Operators.*—Substations that are not adjacent to a power plant usually have a single attendant in each of two or three shifts a day. If, however, the substations are very large, so that more than one man is required during emergency periods, more men will be employed.

(*d*) *Power Development Maintenance and Repairs.*—These may be divided into two classes, i.e., inside and outside maintenance and repairs.

Inside maintenance and repairs generally vary from about 0.05 mill per kilowatt-hour of generated energy for very small plants, to about 0.025 mill per kilowatt-hour for the very largest plants.

The cost of outside maintenance and repairs is usually not very large for the best types of structures unless there are errors in workmanship or design. It depends greatly upon the character of the development, but about 0.1 per cent of the construction cost of the outside items of the development is a reasonable figure.

(*e*) *Transmission-line Maintenance and Repairs.*—This may be assumed to be about 0.5 per cent of the construction cost of the transmission lines.

(*f*) *Substation Maintenance and Repairs.*—This may be assumed to be about 0.2 per cent of the construction cost of the substations.

(*g*) *Taxes.*—This varies with the locality, and data can be easily obtained for each case.

(*h*) *Insurance.*—The usual insurance policies carried are fire, workmen's compensation, and public liability. Fire insurance is generally applicable only to the power house and contents and to outside transformers and switches, etc. The annual rate is about \$3.10 per \$1000 unless there are special hazards.

Workmen's compensation and public liability insurance are usually both based on the annual payroll of the employees engaged in operating the plant, transmission line, etc. The annual cost of these two policies amounts to about 11 per cent of the operating labor costs, and for approximate estimates, about 11 per cent of the sum of the foregoing items (*a*) to (*f*) inclusive, may be assumed.

(*i*) *Depreciation Reserve.*—This item is discussed in Sec. 473.

(*j*) *Management, Supervision and Home Office Expense.*—This varies widely with the size of the system, the character of customers served, and the nature of the organization. That part of such overhead charges which is applicable strictly to power-plant operation and the expense of selling wholesale power, not including the cost of investigations and negotiations for extensions of the system, should not exceed about 0.2 mill per kilowatt-hour of generated energy for very small plants or 0.05 mill per kilowatt-hour for very large plants.

(*k*) *Interest Charges.*—This item is discussed in Sec. 458.

In the foregoing discussion, the annual charges for city distribution systems have not been considered. This is an extremely specialized subject and is outside the scope of this book.

473. Annual Depreciation.—The annual depreciation of any structure or piece of apparatus is literally its annual loss in value. Actually an item may be as useful and efficient ten years after its installation as when new. However, it has depreciated in value as the extent of its useful life has decreased.

As used in Item 15 of Table LXXXI and elsewhere, the "annual depreciation reserve" is the amount of money which must be set aside each year at compound interest to provide a fund which will be adequate for all necessary replacement of worn-out equipment and structures.

Some engineers include in this reserve an amount which would be sufficient to replace apparatus that has become obsolete before it has worn out. This is rational if based on the assumption that improvements in equipment will be such that replacements will be necessary on the ground of competition with improved new equipment used in competing systems. However, this feature has never been a serious one, and replacements due to obsolescence have been practically always in the nature of an added investment, the return on which is provided by increased efficiency of operation. Therefore, an allowance for obsolescence has seldom been included in estimates of the annual depreciation reserve.

While the theory upon which annual depreciation is based is generally agreed upon, the question of probable life of the various items of the development is subject to large differences of opinion. This is necessarily so because the period of useful life is speculative and depends upon the care with which the plant will be operated and the extent of maintenance. For instance, a timber dam, if neglected, will become useless in a few years, while cases have been cited where such dams were eighty years old and still in use. However, in such very old dams, few of the original members remain; and the cost of replacing such members from time to time, together with that of small parts

of the dam which have actually failed, has been charged to maintenance. Thus it is seen that it is difficult to define where maintenance charges stop and replacement reserve charges begin, and as the probable life is a matter of judgment, it is not to be expected that engineers will agree closely.

The depreciation reserve, together with proper maintenance, is supposed to keep the original value of the plant in perpetuity, in which case no provision for an additional fund to amortize outstanding securities is necessary. When a bond issue becomes due, the plant will still have a value which will be ample security for a new issue.

Table XCII gives the probable life of various parts of a hydro-electric development, as used by the writer. It assumes first-class construction, particularly for exposed concrete work in cold climates, and reasonable maintenance.

TABLE XCII

PROBABLE LIFE OF STRUCTURES AND APPARATUS

Item	Life in Years	Item	Life in Years
Steel Bridges	25	Concrete Substructure	*
Earth Dams	*	Brick and Steel Superstructures	50
Concrete Dams	*	All Power House Equipment	20
Timber Dams	20	Wood Pole Lines (Copper cost about one-third total cost)	15
Canals	*	Steel Tower Lines (Copper cost about one-third total cost)	30
Wooden Flumes	10	Outdoor Substation Structures.	25
Steel Flumes	20	Outdoor Substation Apparatus.	20
Concrete Flumes	40	Frame Dwellings	25
Tunnels	*	Metal Gates and Valves:	
Steel Pipe:	20	Low-velocity	20
Open	25	High-velocity	10
Buried	20	Timber Gates	10
Wood Pipe (see also Sect. 241).	20	Exposed Gate Hoists	15
Concrete Pipe	40		
Racks	15		
Rack Structures	20		

* These items are expected to be kept in perpetual usefulness by proper maintenance and repairs.

Table XCIII shows the equal annual payments to a sinking fund necessary to accumulate one dollar at the end of a given number of years, based on interest on the sinking fund being paid annually. The table is derived from the following equation:

$$A = \frac{i}{(1+i)^n - 1}, \qquad \qquad (245)$$

where A = each annual payment, in dollars;

i = the interest rate, expressed as a decimal, and

n = the period of years.

If interest is paid semi-annually, the annual payments will only be slightly less.

TABLE XCIII

ANNUAL PAYMENTS REQUIRED TO ACCUMULATE ONE DOLLAR AT END OF A GIVEN NUMBER OF YEARS

Interest Compounded Annually

Number of Years	Interest Rate					
	4%	5%	6%	7%	8%	9%
2	0.497	0.488	0.485	0.482	0.480	0.478
3	0.320	0.317	0.314	0.311	0.309	0.305
4	0.235	0.232	0.228	0.225	0.222	0.218
5	0.184	0.181	0.177	0.174	0.171	0.167
6	0.151	0.147	0.143	0.140	0.136	0.133
7	0.126	0.123	0.119	0.115	0.112	0.109
8	0.109	0.105	0.101	0.098	0.094	0.091
10	0.0834	0.0795	0.0758	0.0724	0.0690	0.0658
12	0.0666	0.0628	0.0592	0.0559	0.0527	0.0496
15	0.0499	0.0463	0.0429	0.0398	0.0368	0.0341
20	0.0336	0.0303	0.0272	0.0244	0.0218	0.0195
25	0.0240	0.0209	0.0182	0.0158	0.0137	0.0118
30	0.0178	0.0150	0.0127	0.0106	0.00883	0.00734
35	0.0136	0.0111	0.00900	0.00724	0.00580	0.00464
40	0.0105	0.00828	0.00647	0.00501	0.00377	0.00296
45	0.00826	0.00627	0.00470	0.00350	0.00259	0.00190
50	0.00655	0.00478	0.00344	0.00246	0.00175	0.00123

Let us assume that a given item, which has an estimated life of twenty years, would cost $10,000 to replace. In calculating the depreciation reserve, it is usual to assume a rate of interest equal to that used in calculating the annual interest charges (see Item 26 of the financial statement of Table LXXXI), on the theory that the reserve can be used to buy back the company's own securities or can be reinvested in the company's new projects at that rate of return. Therefore we shall assume a rate of interest of 7 per cent.

From Table XCIII it is seen that an annual payment of $0.0244, with interest at 7 per cent compounded annually, will accumulate $1 at the end of twenty years. Therefore, to accumulate $10,000 will require an annual payment of 2.44 per cent of $10,000 or $244 which is the annual depreciation reserve for that item.

The depreciation reserve for any item of the development must not be based solely on the estimated first cost of that item. It must be sufficient for the cost of removal of the item, taking care of adjacent structures, loss of revenue during replacement, and the cost of replacement. As these items are difficult to estimate, it is only necessary to make a liberal allowance. It is permissible to credit the reserve with the probable scrap value of the item, but this is usually neglected as inconsequential.

474. Accrued Depreciation.—In the preceding section, the " anual depreciation " of an item was defined as its annual loss of value. " Accrued depreciation " may be similarly defined as its accumulated loss in value at the end of a given period of years equal to or less than its estimated life.

Also in the preceding section, the "annual depreciation reserve," in dollars, was defined as the annual payment to a replacement fund. "Accrued depreciation reserve," as herein used, may be similarly defined as the accumulated replacement fund at the end of a given period of years.

The derivation of the amount of accrued depreciation reserve may be explained best by an example. In the example of the preceding section, it was shown that, to provide a fund of $10,000 for the replacement of an item at the end of twenty years, it was necessary to set aside annually 2.44 per cent of $10,000, or $244 at 7 per cent compound interest. Suppose it is desired to know the accrued depreciation when the item is ten years old. From Table XCIII it is seen that $0.0724 set aside annually at 7 per cent compound interest will accumulate $1 at the end of ten years. Therefore an annual deposit of $244 will, at the end of ten years, amount to $244 \times \frac{1.00}{0.0724} = \$3370$, which is the accrued depreciation of the item at the end of ten years.

The foregoing method of computing accrued depreciation is known as the "sinking-fund" method and is generally used by engineers in reports. It will be noted, in the previous example, that the item in question, having passed through one-half its estimated life, is only 33.7 per cent depreciated. This is because the reserve accruals with compound interest will be more rapid in the latter part of its life than in the first part.

The actual physical depreciation not being determinable, the sinking-fund method of estimating it has the most favorable features from many points of view. However, some commissions having jurisdiction over the accounting of public service corporations have decreed that accrued depreciation, as affecting statements of capital account, shall be figured by what is known as the "straight-line" method. This method is based on the assumption that accrued depreciation is directly proportional to the age of the item. In other words, the accrued depreciation at a given time is equal to its first cost times the ratio of its age to its estimated life. Thus an item, having passed through one-half its estimated life, is 50 per cent depreciated.

While engineers usually estimate accrued depreciation on the basis of the estimated cost of replacement, and not the first cost, as explained heretofore, commissions and accountants generally use the first cost only, assuming that if replacement cost is in excess of first cost, the difference is a charge against operation.

475. Legal Requirements.—The engineer should obtain competent advice on Federal and local legal requirements that affect the value of a plant or project. Such requirements are too numerous to describe here, and differ materially according to location and the company's charter rights.

The Federal Power Commission has jurisdiction over all developments on navigable streams and those requiring the use of government lands. As "navigable" streams embrace not only those that are at present navigable, but those that can be made so by the construction of locks and dams, the jurisdiction of the Commission is theoretically unlimited and the practical limit has not been well defined. For such developments, permits must be requested and licenses obtained, and all plans are subject to the approval of

the Commission. Certain fees and rentals are demanded and locks are sometimes required.

Several usual legal requirements are mentioned in Item VIII of Table XC. One state prohibits the exportation of power; another requires reservoirs and ponds to be cleared of trees and brush to the bottom instead of to low water; many states require dams to be built in accordance with certain specifications which are quite radical; cemeteries may be condemned for flowage in some states and not in others; state lands, in many states, may not be used for a development under any circumstances, and in others, only by an act of legislature.

The right to interfere with the flow of the stream, to the detriment of other users on the stream, exists only under special circumstances. Therefore a project may not be allowed to operate at a low load factor, if there are plants below that have no pond and are equipped to run only at a high load factor and therefore require a fairly continuous flow.

These and other requirements and limitations have been sufficient, in some cases, to prevent the development of otherwise excellent projects.

476. Competition.—The probability of successful competition from other sources of power in the command of unfriendly interests must receive consideration. It has been pointed out that the rate of return from an investment in a project must at least equal that which may be obtained from other available similar projects or plants. For this reason it is necessary for the engineer to study the other possibilities of producing power in the vicinity. It is not always possible to investigate the field of competition to as great an extent as is desirable. Fortunately it is seldom likely that, in a limited market, a competitive plant will be developed even if slightly better. Such a procedure could only result in a rate war which would probably ruin both enterprises. However, it is well for the engineer to formulate an approximate opinion of the cost of producing power from other possible sources in the vicinity.

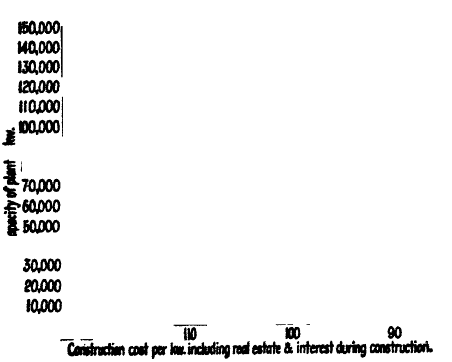


FIG. 479.—Construction Cost per Kw. Various sizes steam electric power plants including real estate and interest during construction, 1925.

The chief competitor of a water-power project is usually a steam plant. The following charts, prepared by E. R. Welles, Mechanical Engineer, The J. G. White Engineering Corporation, show the approximate cost of producing energy from steam plants under various operating conditions. No transmission costs are included.

It should be borne in mind that conditions which arise in the cost of building and operating steam electric power plants vary so widely that it is impossible to prepare very accurate figures without a study of the

particular case. The charts should only be used as a guide for the purpose of arriving at the approximate costs. When a very definite situation arises in which the plant location, load conditions, water supply, and fuel supply, can be determined, a more exact estimate should be made to determine what the results would be. These charts are described in detail below:

Figure 479. *Construction Costs per Kilowatt. Various Sizes Steam Electric Power Plant.*—This chart shows two curves, one indicating the maximum cost of construction and the other the minimum. Capacities are given from 10,000 to 150,000 kw. and the construction cost is shown in dollars per kilowatt, including real estate, contingencies, engineering and construction fees, and interest during construction.

Figure 480. *Fixed Changes per Kilowatt-hour at Different Load Factors* (*Maximum Cost*).—This sheet is derived from Fig. 479, using the maximum

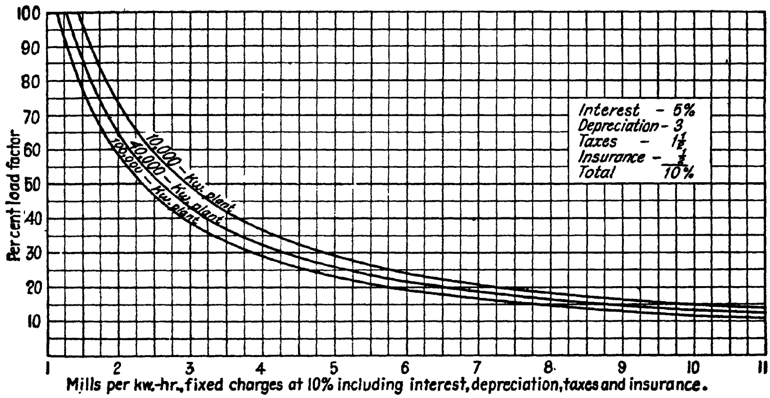


FIG. 480.—Fixed Charges per Kw.-Hr. at Different Load Factors. Various sizes steam electric power plants based on maximum construction cost.

cost per kilowatt for construction. Three curves have been prepared, one representing a 10,000 kw. plant, one a 40,000-kw. plant and one a 100,000-kw. plant. The cost in mills per kilowatt-hour has been computed for fixed charges at 10 per cent including 5 per cent interest, 3 per cent depreciation, 1½ per cent taxes and ½ per cent insurance. It is realized that 10 per cent is too low a figure in most cases for this charge, but it is used for the reason that it makes it a simple matter to derive the actual cost per kilowatt-hour, when the fixed charge percentage is known, by the addition of a percentage to the figures given. If the plant is to run only a part of the year, the costs per kilowatt-hour should be divided by the percentage of time the plant is run.

Figure 481. *Fixed Charges per Kilowatt-hour at Different Load Factors* (*Minimum Cost*).—This sheet is the same as Fig. 480, except that the minimum construction costs are used.

Figure 482. 10,000-*kw. Steam Electric Power Plant, Cost of Power Including Fuel, Labor and Maintenance.*—This sheet gives the cost for generation, including fuel, labor and maintenance in a 10,000-kw. plant. The curves

indicate the cost with coal at various prices and are meant to represent bituminous coal having a heating value of approximately 14,000 B.t.u. per pound, one ton being 2000 lb. The costs are given at various load factors and are shown in mills per kilowatt-hour for fuel, labor and maintenance. In the event anthracite coal is considered, using No. 3 Buckwheat, approximately 25 per cent should be added to the kilowatt costs to the corresponding costs per ton of bituminous coal, this also being based on a ton of 2000 lb. Fuel oil at $1 a barrel is approximately equivalent is these cases to bituminous coal at $4.20 per ton.

It should be borne in mind particularly that it is an extremely difficult matter to compute a cost per kilowatt-hour for the various load factors, inasmuch as there may be a considerable difference in economy with different characters of load curves, although the load factor might be the same. The

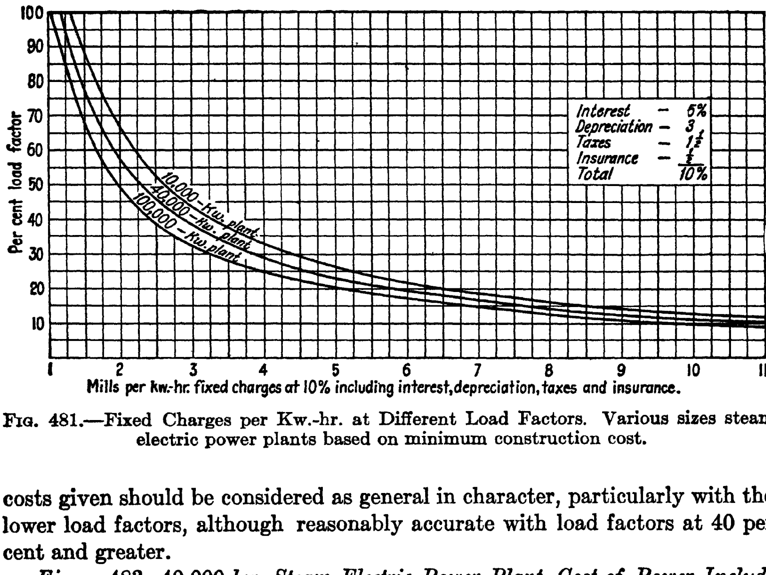


FIG. 481.—Fixed Charges per Kw.-hr. at Different Load Factors. Various sizes steam electric power plants based on minimum construction cost.

costs given should be considered as general in character, particularly with the lower load factors, although reasonably accurate with load factors at 40 per cent and greater.

Figure 483. 40,000-*kw. Steam Electric Power Plant, Cost of Power Including Fuel, Labor and Maintenance.*—This sheet is the same as Fig. 482, except that it is for a 40,000-kw. plant.

Figure 484. 100,000-*kw. Steam Electric Power Plant, Cost of Power Including Fuel, Labor and Maintenance.*—This sheet is the same as Figs. 482 and 483, except that it is for a 100,000-kw. plant.

As an example, to indicate the use of the foregoing charts, suppose it is desired to know approximately the cost of producing power from a 40,000-kw. steam plant at 40 per cent load factor with total fixed charges at 12 per cent. Coal is assumed at $6 per ton.

From Fig. 479, the maximum probable construction cost for a 40,000-kw. plant will be 111 × 40,000 = $4,440,000.

From Fig. 480, which is for maximum costs, the fixed charges for a 40,000-kw. plant operating at 40 per cent load factor, are 3.2 mills per kilowatt-hour, based on a 10 per cent rate. As the rate is to be 12 per cent, the fixed charges will be $\frac{12}{10} \times 3.2 = 3.84$ mills. (Incidentally, if the plant is to run for only one-quarter of a year at this load factor, the fixed charges per kilowatt-hour will be four times as great).

From Fig. 483, the operating costs of a 40,000-kw. plant, operating at 40 per cent load factor and with \$6 coal, are 7.3 mills per kilowatt-hour.

The sum of the fixed charges (3.84 mills) and the operating charges (7.3 mills) or 11.14 mills per kilowatt-hour represents the probable maximum cost under the conditions assumed. In the same way, a probable minimum cost of 10.72 mills can be obtained by using Fig. 481 instead of Fig. 480, thus providing the two probable extremes between which the actual cost will probably lie.

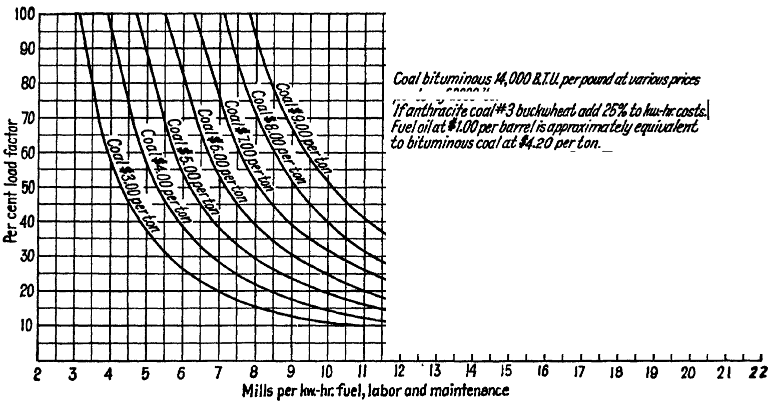


FIG. 482.—10,000 Kw. Steam Electric Power Plant. Cost of power including fuel labor and maintenance, 1925.

Figures 485 and 486, prepared by W. P. Creager, show very approximate 1925 costs of producing energy from hydro-electric developments from 50 to 400 ft. head and for various amounts of flow. Fig. 485 is for 50 per cent and Fig. 486 for $33\frac{1}{3}$ per cent load-factor plants. They should never be used for estimating purposes and have been included herein only after considerable hesitation. They are useful only for the following purposes:

1. To determine whether a project is within the realm of probable usefulness, or possible competition.
2. To assist in determining, quickly but roughly, which of a series of projects is probably the most economical, as when comparative estimates are to be made, and it is desired to start with the one that appears the most attractive.
3. To indicate to the student the approximate relation between head, flow and length of pipe line as affecting cost of energy.

The curves are based on the following assumptions:

1. The output is measured at the switchboard.
2. The development consists of a dam, a wood-stave pipe, a surge tank where necessary, a two or three-unit power house, and a short tail race.

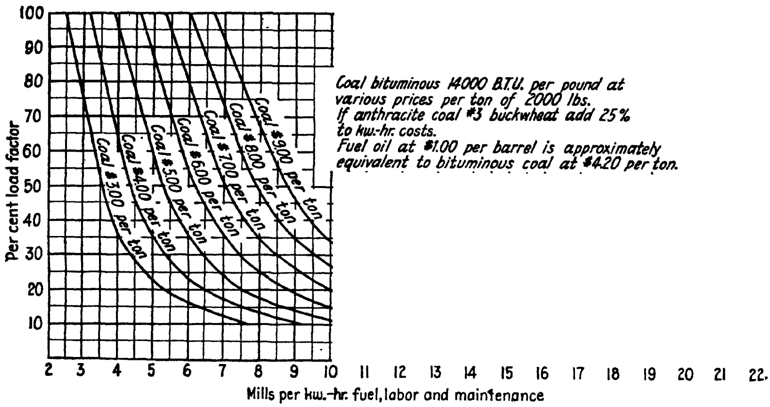


FIG. 483.—40,000 Kw. Steam Electric Plant. Cost of power including fuel, labor and maintenance, 1925.

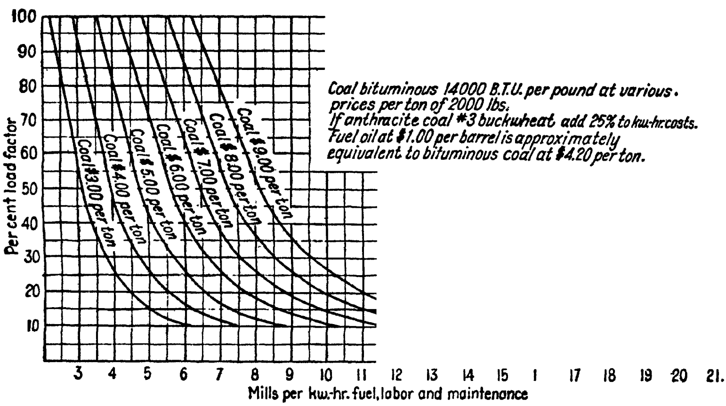


FIG. 484.—100,000 Kw. Steam Electric Power Plant. Cost of power including fuel, labor and maintenance, 1925.

3. The diversion dam, including intake, costs $110,000.
4. The road to the site costs $14,000.
5. Costs include 10 miles of single-circuit, 66,000-volt, steel-tower transmission line.
6. No trunk transmission tolls are included.

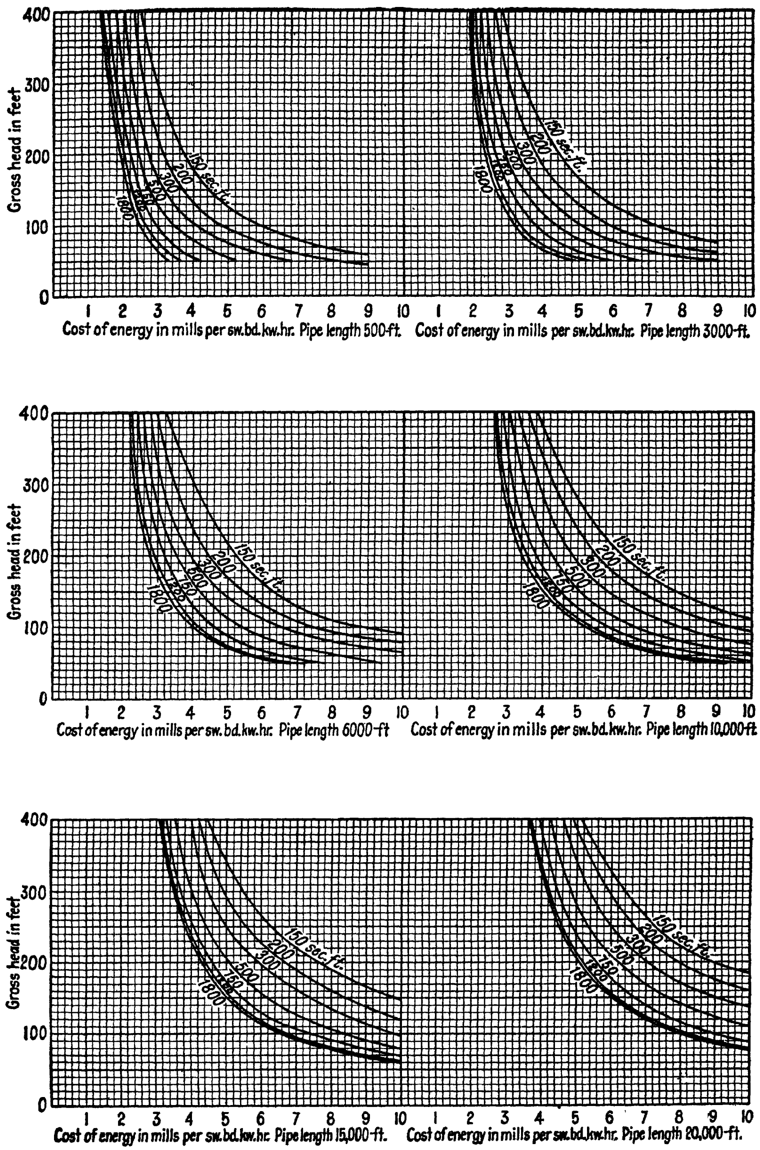


FIG. 485.—Curves Showing a Rough Estimate of Cost of Producing Energy from Hydroelectric Developments Based on Assumptions Described in the Text. Do not use for estimating purposes. 50 per cent load factor.

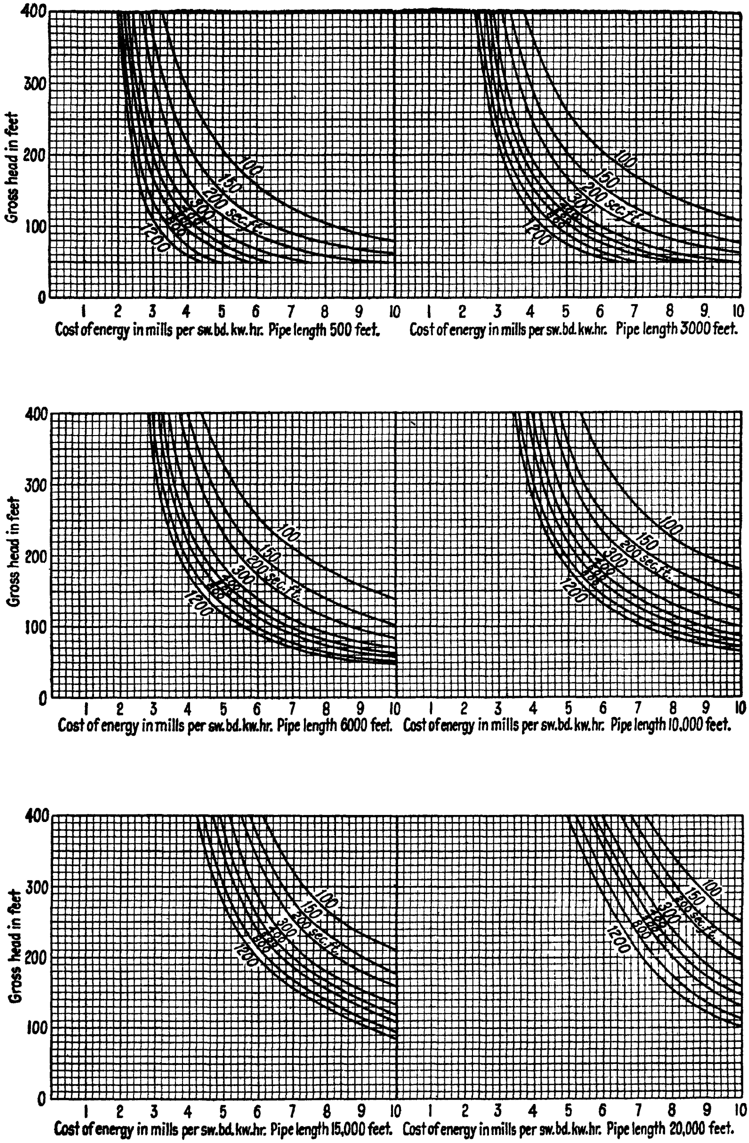


FIG. 486.—Curves Showing a Rough Extimate of Cost of Producing Energy from Hydroelectric Developments Based on Assumptions Described in the Text. Do not use for estimating purposes. 33⅓ per cent load factor.

7. The pond occupies 430 acres.
8. Flowage costs $43,000.
9. Clearing pond costs $43,000.
10. Purchase price of water rights and rights of way for transmission is not included.
11. Annual charges include,

Interest on investment	7 per cent
Taxes and insurance	1 per cent
Depreciation	2 per cent
Storage reservoir charge	1 mill per kw.-hr.

    Necessary operation, maintenance and overhead.
12. All discharges are continuous second-feet flow, and all output is primary.
13. Costs are as of 1925.
14. The heads are gross heads to crest of dam.

Corrections for changes in the assumptions upon which the curves are based can be made readily by dividing the corresponding change in annual charges by the annual kilowatt-hour output determined from the head and discharge, it being remembered that the head is the gross head.

CHAPTER XXXIV

RIVER GAGING

By Nathan C. Grover and John C. Hoyt

477. Introduction.—The following statement, covering the essential principles involved in river gaging, will serve the needs of the engineer who uses records of stream flow in designing and operating hydraulic power plants. For more complete details of many phases of the collection of systematic records of river discharge, reference is made to " River Discharge,"[1] by Hoyt and Grover.

Computation of daily discharge of a surface stream is based on the axiom that the discharge at any given stage will be unchanged so long as the bed and banks of the stretch of river controlling the stage-discharge relation at the gage remain unchanged, and that the discharge will vary with the stage according to some natural but fixed law. This stable relation between stage and discharge makes practicable the construction of a rating (stage-discharge) curve (Fig. 494) from a few measurements of discharge that are well distributed over the range of stage. This rating curve is the graphic representation of the formula for computing the discharge past the gage. Any change in the conditions affecting the stage-discharge relation at the gage will make it necessary to construct a new rating curve. If there is no change in these conditions, measurements made in different years will plot on the same curve, which may be applied to a record of stage extending over the whole period to obtain estimates of daily discharge.

The stage-discharge relation will be stable when there exists below the gage an effective " control " section in which the bed and banks are permanent. The control may be found: (1) at the head of rapids where the fall is large; (2) at a succession of two or more riffles at each of which there is a moderate fall; (3) in a long stretch of the channel of a stream having a low gradient; or (4) at an artificial structure in the channel. Natural stability of the stage-discharge relation can be obtained only by careful selection of a site where the bed and banks of the river channel below the gage are so stable that the stage-discharge relation does not change because of erosion or deposition of silt or drift. Obviously, a site should not be selected above a dam or other structure containing openings through which the flow is controlled by gates that may be open or closed, so that the stage is not a true index of discharge.

The sensitiveness of the stage-discharge relation is important. A station

[1] Published by John Wiley & Sons, Inc.

should be selected so as to give as large a change in stage as is possible for a given change in discharge. Estimates of flow at stations that have relatively small fluctuations in stage are liable to large errors on account of lack of refinement in the determination of mean daily stage. The rate of fluctuation of stage at the gage should be such that the least change in discharge to be recorded, say 1 per cent of the total flow, should cause a readable change of stage. The sensitiveness will be determined by the shape of the control. A broad, flat control will cause a small fluctuation in stage for a large change in discharge, and a notch-shaped control will cause a large fluctuation in stage for a relatively small change in discharge.

The stage-discharge curve for an ordinary gaging station is not greatly different in appearance from the discharge curve for a weir. Although parabolic in form, it may not be a simple parabola like that for a weir, as the section or sections that determine the relation of stage to discharge are irregular in shape. As indicated above, however, the stage-discharge relation for any river channel of permanent control-section will be as fixed as that for the flow over a weir. The curve may be accurately defined by a few current-meter measurements of discharge, and, when it is so defined, records of stage may be converted into records of discharge without appreciable error.

The data to be collected in the field therefore consist of records of daily stage of the stream and sufficient measurements of discharge to define accurately the stage-discharge curve.

478. Establishment of Gaging Stations.—The selection of a satisfactory site for a gaging station involves finding (1) a site for the gage where the stage-discharge relation will be stable and (2) a section or sections for making the discharge measurements in which the current is smooth and the velocity is measurable—that is, between 0.5 and 15 ft. per second—at all stages of the river. If time permits, all possible sites for the gage should be inspected at high as well as at low stages and in both winter and summer, before final choice is made and the gage installed.

Accessibility of a site is important but is not a controlling condition. Over a period of years, a good record of discharge will cost less at a site that is expensive to reach but has a stable control than at one that is less expensive to reach but has an unstable control. Similarly, the availability of a local caretaker for the gage is important but not essential. Recording gages can now be furnished that will record the stage accurately for sixty days or more without attention, thus practically eliminating the necessity for a local observer.

The selection of a site where there is a bridge available for supporting the engineer while he is making observations of depth and velocity may be desirable as a matter of economy in equipping the station, but few bridges occupy the best sites for making measurements of discharge. Moreover, bridge piers disturb the smoothness of the current and therefore decrease the accuracy of the determinations of velocity. Hence some degree of accuracy is generally sacrificed when a bridge is chosen as a site for making gagings. The erection of a cable and car outfit for use of the engineer in measuring discharge involves considerable expense, but it may be more than justified because it yields more accurate records.

The site for the gage itself is most important. Whether it is of the recording or non-recording type, the gage must be accessible at all stages and must be so situated that it will not be unduly exposed to danger of damage from drift. If possible the gage must be so placed with respect to normal currents that drift or silt will not be deposited in such a manner as to cut off its open connection with the free surface of the flowing water. The most critical section of a river related to a gaging station is therefore that section for which the stage-discharge relation is to be determined, or the section in which the gage is placed, and not necessarily that in which mesaurements of discharge are made. Such measurements are frequently made in some other section than that of the gage, and not uncommonly different sections are used for measuring the flow at different stages of the river, in order to obtain the highest degree of accuracy. Where the flow is steady—that is, neither rising nor falling—the same quantity of water is passing each section of the river, and measurements of discharge made in other sections apply equally well to the section of the gage. The essential requirement is stability of the section or sections controlling the stage-discharge relation at the gage, and the accuracy of records will not be affected by shifts in bed and banks, either in the section of the meter measurements or in the gage section, provided such shifts do not extend to the control section and cause changes in the stage-discharge relation at the gage. This statement may be illustrated by comparison with a weir. The relation of head on the weir to the quantity of flow over the weir is controlled by the shape and height of the crest and is not affected by changes in the bed or banks of the pool above the weir if those changes are not sufficiently great to affect materially the magnitude or distribution of the velocities of the approaching water. Accurate current-meter measurements of discharge made within the pool would plot on the discharge curve of the weir even if considerable changes in the bed or banks or both occurred between successive discharge measurements. The pool above a dam or weir is not, however, a good site for a gage, even if the dam and its foundations are tight and no water is discharged through or around the dam, because at such a site the relation of stage to discharge will not be sensitive at low stages. In order that any method involving the conversion of observations of stage into records of discharge may be satisfactorily applied, the percentage of flow that causes a readable increment or decrement of stage must not exceed the percentage error allowable. For example, if a record is desired that is accurate within 1 per cent, a single second-foot of water must make a readable difference in stage when the flow is 100 sec.-ft. This degree of sensitiveness cannot ordinarily be obtained in a pool above a dam with a level crest.

479. Equipment of Station.[2]—The site having been selected, the establishment of the gaging station consists of the installation of a gage for observing or recording stage, and a cable or other structure from which the measurements of discharge may be made (Fig. 487).

The gage must be so placed that it may be read easily and accurately at all stages of the river and that its datum will remain constant. It must be

[2] Plans and specifications for equipment for gaging stations are available at the United States Geological Survey.

referred to bench marks entirely removed from the gage or the structure on which it is placed, in order that the gage, if disturbed, may be replaced to the same datum. The gage should be graduated and should generally be read to hundredths of a foot at low stages of the river and to tenths or half-tenths of a foot at high stages.

The type of gage used will depend on many factors. A staff gage, either vertical or inclined, is perhaps the most common. Such a gage may be painted directly on wood, or preferably, if vertical, it may be stenciled and enameled on sheet metal, which should be attached to a plank or timber.[3] It must be rigidly attached to solid rock, a masonry bridge pier or abutment, or the foundation wall of a building at the water's edge. It should not be attached to a tree at the water's edge, because of danger that the datum of the gage may be changed if the roots of the tree are undermined or that the

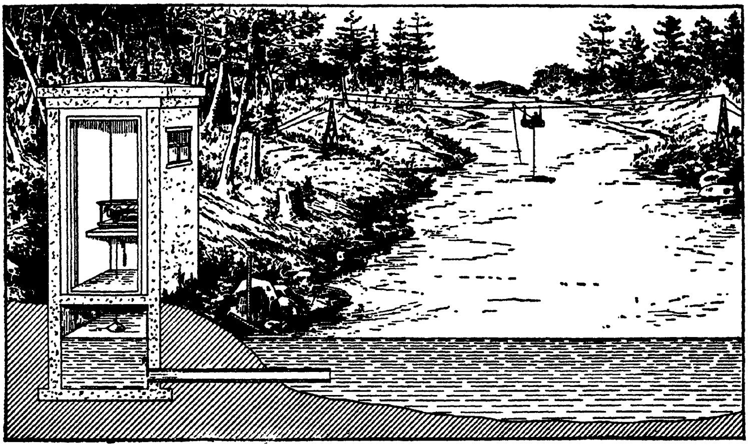


FIG. 487.—Typical Cable Station with Automatic Register.

gage may be lost if the tree is carried away at high water. If attached to wooden piling the datum must be carefully watched, especially if the river has a heavy ice cover in winter. If no suitable rock or structure is available, the gage may be attached to masonry piers built for the purpose. Such piers should have their foundations below the frost line and be otherwise so constructed as to be in minimum danger of heaving. An inclined staff gage must be graduated by use of a Y level after it is placed in final position, and, as no two inclined gages will have exactly the same slope and therefore the same length of graduation, the use of enameled-metal facing for such a gage is not practicable. An inclined gage may be made up of two or more sections of different slope.

Staff gages are difficult to maintain in rivers subject to heavy ice cover, to large quantities of drift, or to log "driving." In such rivers chain gages are

[3] Enameled gages may be obtained from the Baltimore Enamel Co., Baltimore, Md.

frequently used because they are entirely removed from danger of damage by ice, logs, or other drift. The chain gage (Fig. 488) consists of a graduated scale board, 10 ft. or more in length, extending from a box supporting a pulley wheel, over which runs a heavy sash chain with a weight attached at one end and a marker (*M*) near the other end. This apparatus is fastened in a horizontal position to a bridge or other structure, so that the weight when lowered will come into contact with moving water, as the stage will be indicated by the position of the marker against the scale board when the weight just touches the water, and the exact point of contact cannot easily be determined by the observer above if the water is still. When not in use the chain is drawn up and placed in the box. If the range of stage is greater than the length of the

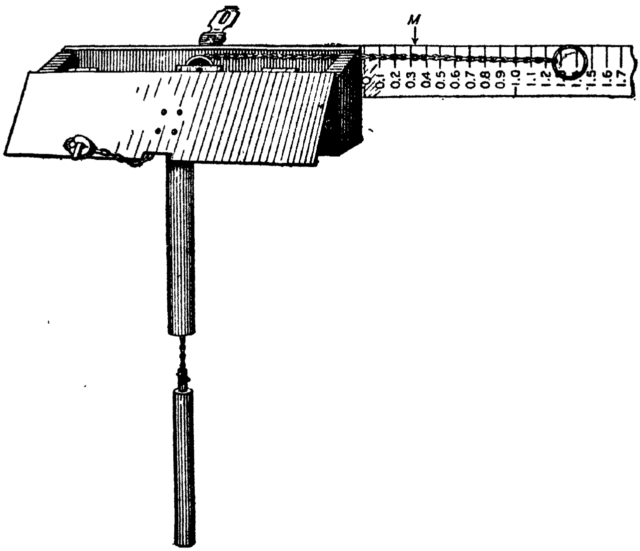


FIG. 488.—Chain Gage.

scale board, it is necessary for the chain to carry a second, and for extreme ranges a third marker spaced at intervals of 10 ft. from the first marker. The reading indicated by these supplementary markers is, of course, 10 or 20 ft. more than the figures shown on the gage. The distance from the bottom of the weight to the principal, or low water marker,—the chain length—should be measured frequently with a steel tape and maintained at a constant length to overcome errors in the records resulting from stretch or wear of the chain. The chain gage would be damaged or lost only by failure of the structure to which is attached.

The installation of an automatic register of the float type (Fig. 487) involves a table that is rigidly set for supporting the register and that will not shift in elevation and is above the highest stage of the river; a float well, generally and preferably vertical, so connected with the flowing water of the river

that the stage of water in the well, which is, of course, the stage recorded, will at all times be the same as the stage of water in the river; a shelter to protect the gage from rain and dust and from inquisitive or malicious molestation; and, in regions of frost, such protection for the well, or in some localities such provision for heating the well, as to remove the danger of formation of ice around the float. The gage shelter and well must be so situated as to be free, if possible, from danger of destruction by ice, logs, or other drift; and so constructed as to withstand the strains of such pressure and knocks as cannot be avoided. As the gage costs from $150 to $250 and its installation costs from two to ten times as much, the cost varying with the site, considerable care and expense are warranted in avoiding the danger of destruction or loss.

If these fundamental requirements are followed, many entirely satisfactory styles of wells and shelters may be built. The supporting table need be only large enough to receive the gage, and it must be perforated to permit the free action of the chains and tapes leading from the register to the float, counterweight, and clock weight. It may rest on the top of the well, if that is solidly built, or on an independent foundation. The well should be large enough to admit a man and permit him to do such work as may be needed, and should have a trap-door entrance from the shelter and preferably one or more entrances from the outside. In climates where there is no trouble from frost, several entrances at different heights are advisable. If built of concrete or wood, the well should be at least 3 by 3 ft. and preferably 4 by 4 ft. in cross-section. It should be equipped with a ladder leading from the trap-door to the bottom and solidly attached to one wall or built as an integral part of it. Both corrugated or other iron pipe and cement pipe have been satisfactorily used for a well, especially in regions of no frost. Such pipe wells should be large enough to permit a man to enter from the top if side entrances cannot be arranged.

The intake pipe should be 3 to 5 in. in diameter, set normal to the current and always fully open for the passage of water, either inward with a rising stage or outward with a falling stage. In clear-water streams there is no difficulty in keeping an intake pipe open. In silt-laden streams special precautions may be necessary. If the topography permits, the intake pipe may be omitted and a direct connection made between the well and the river. Where this is not possible, some special apparatus may be needed to keep the intake pipe open. Such an apparatus may consist of an endless chain that may be drawn back and forth through the pipe at any stage; of a shut-off valve in the intake pipe and a pump for filling the well, while the valve is closed, with water, which will rush out and clear the pipe when the valve is opened; or, where a supply of clear water from a spring or tributary is available, of a provision for a constant inflow of clear water, which at all times will cause an outward current through the intake pipe.

Generally, float wells on silt-laden streams become filled more or less rapidly with a deposit of silt, which must be removed either through the trap-door in the well-house floor or through a door or doors in the side wall of the well. The silt must be removed so promptly and effectively that the graph record of stage made by the recorder shall be accurate and complete.

The shelter may vary from a relatively small box cover for the gage in an area of dry, warm climate, where there is little danger of frost within the well or of rain at periods when the gage must be opened for winding the clock and changing the record sheet, to a small house 4 by 4 ft. in cross-section and 6 or 7 ft. high, or, in remote regions, to a house large enough for a man to stretch out on the floor and sleep.

The cost of installing a recorder will vary widely with the range of stage of river to be provided for, the topography of the bank in which the well is to be constructed, the nature of the material to be excavated, the remoteness from sources of supply of materials and labor, and the protection that must be provided against the formation of ice in the well and damage to the well and shelter by ice or other drift. A shallow well, for a range of stage of a few feet, and a small wooden shelter may be built for $200 or even less under favorable conditions. In remote localities installations built of concrete, with wells excavated largely in rock and providing for a 50-ft. range in stage, have cost as much as $20,000 each. The cost of ordinary installations, however, ranges between $300 and $1500 in addition to the cost of the register itself.

In northern latitudes the well must be set some distance into the bank, and the exposed part, if built of boards, must be carefully battened to prevent the formation of ice around the float. If electric current is available the installation in the well and the operation in winter of a small electric heater, or even the constant burning during cold weather of one or two electric lamps, will generally eliminate frost. If electric current is not available, a few inches of kerosene or other mineral oil, varying in depth with the intensity and duration of the period of frost, placed on top of the water in the well, will generally remove the danger of ice.

At each gaging station equipped with an automatic register, there should also be a non-recording gage, generally a staff, either vertical or inclined, which should be read and recorded whenever the station is visited. Such a reading will disclose whether the stage of the water in the well is the same as the stage of the river. If it is not, the cause of the discrepancy, generally clogging of the intake pipe, should be removed, and a full record made of exactly what has been done and the effect produced on the record of stage, which is later to be converted into a record of discharge.

The datum of all gages, including the recording register, should be coincident and should be referred to permanent bench marks that are so situated as to be accessible at all stages of the river and convenient for use in checking the gages by means of a Y level.

If no bridge is available at a point where the conditions are suitable for making discharge measurements, a cable must generally be erected. The use of a boat is ordinarily too costly because, in order to handle it in such a way that satisfactory measurements could be made, a boatman would be required in addition to the engineer. The cable with its supports, anchorages, and car represents a considerable initial cost, from $100 for short spans and low supporting towers to $500 or more for long spans and high towers. As the lives of engineers working from the cable are involved, each part of the equipment must be built with an ample factor of safety, to insure stability even

when unusual strains, due to the catching of drift on the meter equipment, are added to those caused by the weight of the man or men working in the car.

480. Measurements of Discharge.—The methods ordinarily used for determining the quantity of water flowing in open channels are of two kinds—the velocity-area method and the weir method. In the former the quantity of discharge is obtained as a summation of the products of partial areas of the cross-section of the flowing water by the respective measured velocities in such areas; in the latter the discharge is computed by a formula (or obtained from tables computed by a formula) from the observed " head " on the weir and the known dimensions of the weir. The velocity-area method is now almost universally used in river gaging because of its comparative cheapness and its essential reliability and applicability under a wide range of conditions, and because of the excessive cost of constructing and rating weirs or dams for river gaging only and the inaccuracies involved in using any except standard sharp-crested weirs with free overfall.

481. Weir Method.[4]—The weir method involves the use of a formula which contains three factors—area of cross-section, velocity, and a coefficient varying with the type of weir. This method is therefore a velocity-area method, but it differs essentially from other velocity-area methods in that the weir is generally rated—that is the relation between stage and discharge is established—by comparison with the laboratory rating of a model. Its accuracy depends primarily on the faithful reproduction in the field of the conditions that prevailed in the laboratory where the rating was made. A weir rated in place is seldom used for river gaging.

The essential purpose served by a weir used in river gaging is to maintain a stable relation between the discharge and the stage of water above the weir. This relation has been determined for standard sharp-crested weirs with and without end contractions by means of many experiments. The Francis weir formula without end contractions ($Q = CBH^{3/2}$), in which Q is the quantity of discharge in cubic feet per second, C is a coefficient equal to 3.33 for sharp-crested weirs but varying for crests of other shapes, B is the length in feet of the horizontal crest of the weir, and H is the depth in feet of water flowing over the weir) is most commonly used in this country.[5] For convenience in use, tables of discharge for sharp-crested weirs of various lengths have been prepared and are available in many handbooks. Many investigations[6] have been made to determine coefficients for use in the Francis formula for the discharge over dams having various shapes of crest. The results have shown a wide range in the value of coefficients, even for dams having crests of similar shape. Estimates of flow at dams are made yet more inaccurate by leakage and by errors in measuring or estimating the water diverted for use in power plants or through waste and flood gates, log sluices, and fish ladders. The collection of accurate records of flow by the weir method is therefore limited

[4] See Sections 75 and 76.

[5] Horton, Robert E., Weir Experiments, Coefficients, and Formulas: U. S. Geol. Survey Water-supply, Paper 200, 1907.

[6] Idem. Lyman, Richard R., Measurements of the Flow of Streams by Approved Forms of Weirs: Trans. Am. Soc. C. E., vol. 77, p. 1189, 1914. Rafter, George W., On the flow of water over dams: Trans. Am. Soc., C. E., vol. 44, p. 220, 1900.

to the use of sharp-crested weirs, which are feasible of construction only where small quantities of water are to be measured and enough of the head to operate the weir can be sacrificed. Such conditions are not commonly found at or near power plants but are characteristic of irrigation canals and distributaries.

The weir method depends for accuracy on the construction of a weir without leakage, with a crest of permanent shape and elevation, of sufficient length and with abutments of sufficient height to carry the largest quantities of water to be measured without exceeding the limit of depth of flow over the crest for which the weir coefficient has been determined. It must be short enough for the smallest amount of water to be measured, say 1 per cent of the flow, to cause a readable change in stage. Uncertainties are large in assuming coefficients for use with the many forms of dams to be found in rivers. Aside from all other sources of error, there is a possibility of an error of 10 to 20 per cent in selecting the coefficient to be used, and the result is a corresponding percentage error in the computed discharge. Rating curves for dams with crests that are long in relation to the quantity of water flowing over them will lack in sensitiveness and will yield inaccurate results on that account. Moreover, the cost of computing records collected at power dams, complicated by partial discharges through wheels of different sizes and operated with varying gate openings and through waste, flood, and other gates, is frequently much greater than the cost of establishing and operating velocity-area stations rated by current meter. On account of these conditions the use of dams for river gaging is not recommended.

482. Velocity-area Method.—The velocity-area method involves the measurement of cross-sectional areas and velocities of the flowing water. The cross-sectional area of the stream is determined by measurements of width and of depth at points so spaced as to show the shape of the bed and the summation of the partial areas computed from these measurements. Sufficient lead must be used to obtain correct soundings. Velocity may be measured by slope, float, or current meter, each involving a fundamentally distinct process.

483. Slope Measurements.—The determination of velocity by a measurement of slope has been described in Sections 34 and 73.

In measuring discharge by the slope method it is necessary to determine the mean area of cross-section and the slope of the surface of the stream and to observe the roughness of the bed and banks, which will determine the proper value of the discharge coefficient. In making such a measurement a straight channel, 200 to 1000 ft. long, must be selected and measured. In this stretch the slope and cross-section should be reasonably uniform and the bed and banks should preferably be permanent. The slope must be sufficiently large to be measured without a large percentage of error.

Errors in applying the slope method, which may aggregate 25 per cent or more, are of three classes: (*a*) errors in the value of slope used in the formula, due either to incorrect observations of the fall in the water surface, which is generally so small that minor errors in leveling or in estimating the position of the water surface make large percentage errors in the result, or to changes in

slope in the stretch of river used; (*b*) errors due to the selection of wrong values of the coefficient to be used in the slope formula; and (*c*) errors in the area of cross-section, which should be the true average of all cross-sections in the stretch.

Slope measurements of velocity have two practical uses: (*a*) in estimating flood discharge, generally after the crest of the flood has passed and when the data available are the slope and area of cross-section as determined from marks along the banks and a knowledge of general conditions of channel and banks during the flood; and (*b*) in designing canals for which the slope must be determined in order that the channel may carry a certain quantity of water at a given velocity.

484. Float Measurements.—The float method of measuring velocity of water flowing in natural channels is direct, is more accurate than the slope method, and is less accurate than the current-meter method. It may be used without special equipment and in rivers carrying so much drift that a current meter cannot be used. Surface floats are generally used because some substance that may be used as a float is almost always at hand, either on the water itself during floods or on the banks at other times. Generally, where subsurface or tube floats can be obtained, the current meter can also be made available, and the increased accuracy to be obtained by using other than surface floats can be obtained better and at less cost by means of the more accurate current meter. Tube floats measure the mean velocity directly with a coefficient of nearly unity and are used satisfactorily in artificial channels of uniform cross-section. They are not adapted to use in natural channels. Velocities obtained by using surface floats must be reduced to mean velocities by coefficients varying between 0.75 for low stages in shallow channels and 0.95 for high stages in deep channels and averaging about 0.90.

Errors in applying the float method, which may aggregate 15 per cent or more, may occur in three ways: (*a*) in failure to obtain a true average area of cross-section of the stretch of river used; (*b*) in failure to obtain properly distributed observations of velocity; and (*c*) in choosing a wrong coefficient to reduce observed velocities to mean velocities.

485. Current-meter Measurements.—The current-meter method is now most commonly used for measuring the velocity of water flowing in an open channel. It combines reasonable cost and practically universal applicability with reasonable accuracy, and may be used on streams of all sizes and depths with velocities ranging from 0.5 to 15 ft. per second. The measurements may be made from a boat, bridge, or cable, or, if the depths and velocities are not too great, by wading. The velocity of the flowing water is measured at several points in a single cross-section. Each measured velocity is multiplied by the appropriate partial area to obtain a partial discharge. All partial discharges are combined to give the total discharge. Errors may be involved in the rating of the current meter and in the observations of its action, but as the measurements are all made in one cross-section this method avoids the error, common in slope and float measurements, resulting from the averaging of velocities over a stretch in which the area of cross-section is varying. As the current meter under proper conditions of use will measure velocity

A

B

FIG. 489.—Typical Gaging Stations.

A. For wading meaurement. *B*. For bridge measurement.

within a probable error of 2 per cent, the measurements are more accurate than those generally obtained by other methods.

In measurements with a current meter a section of the river must be used in which the water flows smoothly and the velocity is reasonably uniform throughout the cross-section. Measurements should not be attempted in turbulent water, as meters of all types are affected, either accelerated or retarded, by vertical motion of the water. The failure of a current meter to operate accurately in such water is as well established as the failure of the Y level to operate accurately in unsteady air. A cross-section should be sought where the current is reasonably regular over the whole width. Such a section can generally be utilized by means of a cable to support the engineer at medium and high stages and by wading at low stages.

The principal sources of error in current-meter measurements of discharge, which should never exceed 5 per cent and in good work will be held below 2 per cent, are in the rating of the meter, in observations of soundings, in placing the meter in position, in observations of revolutions of the meter, in observations of time, and in the use of insufficient observations of velocity.

Ability to place a meter accurately in position depends largely on the use of sufficient lead to reach the bottom without deflection and of the smallest possible cable for supporting the meter that is consistent with proper strength.

Meter ratings do not change rapidly so long as the meter is not damaged by accident, and material errors in rating between velocities of 1 ft. and 15 ft. per second are rare. In general, especially in measuring low velocities, error will be reduced to a minimum by observing the time (not less than 45 seconds) of some number of complete revolutions of the meter. A stop watch recording to fifths of a second should be used for observing time.

A current-meter measurement of discharge is an essential part of river gaging and is made to determine the position of one point on the rating curve for the gaging station. In order to serve this purpose, both the stage and the discharge must be measured accurately. The gage must therefore be accurately set with respect to its datum and accurately read, at least at the beginning and end of the current-meter work, and once each hour during the progress of the work.

The distance from an initial point on one bank of the stream to the points at which depth is measured to determine area of cross-section must be measured and marked on the bridge or cable from which the measurement of discharge is made, in order that the position of the meter with respect to the initial point may at all times be definitely known and recorded and that the cross-section of the flowing water may be accurately computed and the appropriate velocity applied to each partial cross-section. If a wading or boat measurement is made, a measured and tagged line must generally be stretched across the stream to serve the same purpose.

Sufficient observations of depth and velocity must be made to disclose true area of cross-section and true velocity of water in each partial area. In general, observations will be made at points 1 or 2 ft. apart in streams up to

30 or 40 ft. wide, 5 ft. apart in streams up to 150 ft. wide, 10 ft. apart in streams up to 400 ft. wide, and 20 to 50 ft. apart in wider rivers.

There are several methods of making discharge measurements, varying with the number and position of the points at which velocity is observed. In the vertical velocity-curve method, observations of velocity are made at a sufficient number of depths, not less than five, at each point to define the curve of velocities (Fig. 490) at that point. The mean velocity is obtained by averaging the velocities scaled from the curve or by dividing the area between the curve and its axis, as determined by planimeter, by the depth. This method is used as a standard and may be made as accurate as desired by increasing the number of points of measurement. It requires too much time, however, for ordinary use, especially on streams that are of considerable size or are likely to fluctuate in stage and discharge during the measurement.

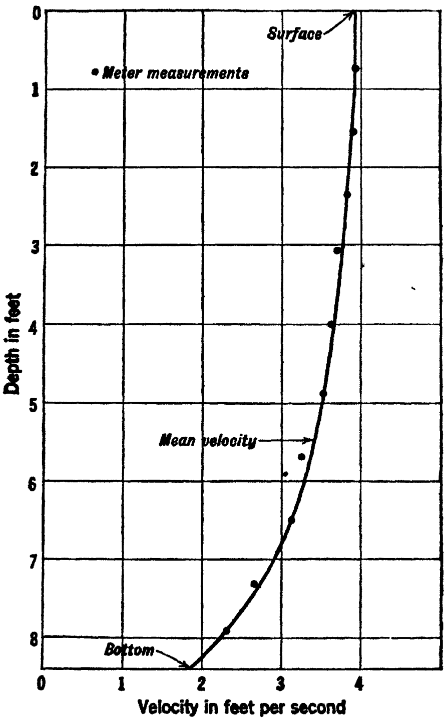


FIG. 490.—Typical Vertical Velocity-curve.

Vertical velocity curves show that the distribution of velocities in the vertical varies as the ordinates to a parabola, and that the velocity at 0.6 depth and the average of the velocities at 0.2 depth and 0.8 depth are very nearly the average of all velocities in the vertical. For this reason it is possible to measure approximately the average velocity in any vertical by one observation at 0.6, or more accurately by two observations at 0.2 depth and 0.8 depth.

After the measurement is completed, the pivot bearings of the meter should be wiped dry and oiled before the meter is returned to its box. As the pivots are made of hardened steel they will rust if not properly cared for, and the rating of the meter will be changed.

If the stage of water is unusual or if the gaging station is not readily and cheaply accessible, the meter measurement should be computed before the station is left. If the plotting of the discharge indicates either change of rating of the station or error in the measurement, the current-meter work should be repeated with great care in order to establish beyond doubt a point on the rating curve.

486. Current Meters and Accessory Equipment.—Several types of meters are used for special conditions of river measurement. The Hess [7] and the Ott [8] direct-action meters have been designed for use in turbulent water and have limited use in testing water wheels and similar special work where smoothly flowing water cannot always be found. The Fteley direct-action meter [9] was designed for use in sewers and other rather small channels and has a reputation for a high degree of accuracy within its field of use. The Haskell direct-action meter [10] was designed for gaging larger rivers and has been widely and successfully used.

The small Price differential meter [11] may be regarded as the universal meter, because it alone satisfies the requirements of general regional river gaging. When equipped with a penta-head, which is a device for indicating every fifth revolution, the meter may be used for measuring velocities ranging from 0.5 to 15 ft. or more per second. It may be operated from a boat, bridge, or cable or by wading, for measuring the discharge of rivers of depths ranging from 0.5 to 50 ft. or more. It may be operated by one man, who also keeps the notes, and may be carried with all accessories by one person traveling on foot or horseback. It is not easily damaged, and with reasonable care does not change in rating. There may be other meters that are somewhat more accurate for measuring turbulent water, but no other meter has yet been developed for universal use. Effort is constantly being made to produce a meter that will measure accurately velocities less than 0.5 ft.

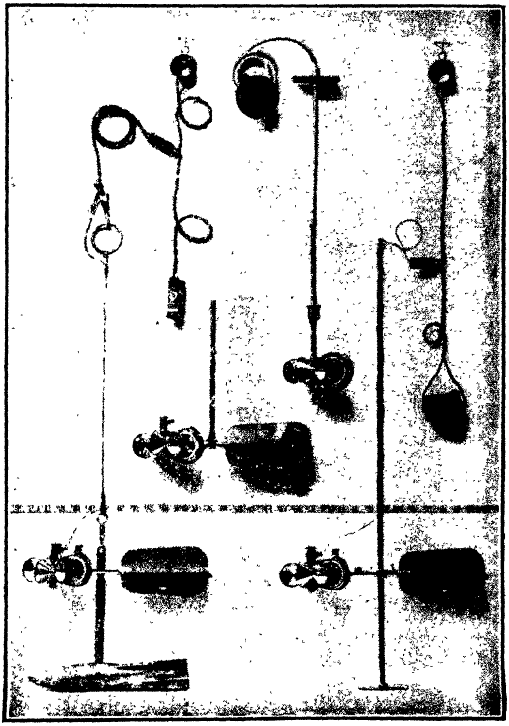


Fig. 491.—*Price Current Meters.*

[7] Manufactured by W. & L. E. Gurley, Troy, N. Y.
[8] Manufactured in Germany.
[9] Manufactured by Buff & Buff Mfg. Co., Boston, and by C. L. Berger & Sons, Boston.
[10] Manufactured in Ithaca, N. Y.
[11] Manufactured by W. & L. E. Gurley, Troy, N. Y.

per second and that may be used reliably in turbulent water. Such a meter must be adapted to the requirements of general use, as indicated above, before it can supplant the small Price meter for this work.

The current meter must be equipped with a device for indicating revolutions of the meter wheel. A sounder is preferred to a recorder, because it makes any failure to act or any irregularity in action of the meter immediately apparent to the operator, who can then correct the difficulty and thus avoid error in the measurement of discharge. A meter that is sufficiently sensitive for measuring low velocities satisfactorily must be designed to indicate only every fifth revolution, in order that the revolution may be countable when the meter is used in currents of high velocity. In general, an electric buzzer operated by a small dry-cell battery is used.

A 25-ft. " metallic " or linen tape is needed for measuring the depth of water as indicated by the length of meter cable paid out in lowering the meter from the surface of the water to the bottom. A steel tape is too fragile for this work.

Lead weights of stream-line shape, ranging from 10 lb. for low velocities and shallow streams to 100 lb. for swift, deep streams, are needed for sounding and for holding the meter in position. Weights heavier than 30 lb. must generally be operated by means of a simple but strong reel, which may be attached to the cable car or to a special rolling crane operated on a bridge floor.

487. Rating of Current Meter.—Current meters are rated by drawing them through still water at known or observed velocities and counting the revolutions. Although meters of the same kind have essentially the same rating, individual meters vary from the average by small percentages. Each meter must therefore be tested and its rating determined. This is done by the United States Bureau of Standards, Washington, D. C., for a small charge, and manufacturers will deliver their meters through the Bureau of Standards in order that the purchaser may obtain a true rating at a minimum cost in time and money.

488. Records of Stage.—The simplest records of stage are obtained by direct readings of the height of the water on a graduated staff. A record of this type will afford a satisfactory record of flow if read once or twice daily on a stream having slow and regular fluctuations, but to ascertain the flow of a stream that fluctuates rapidly in stage, such as most streams used for power, a continuous record is needed. Such a record can be made by an automatic water-stage recorder, which shows all fluctuations of stage by a continuous graph.

Automatic recorders commonly used in gaging rivers record the stage in the form of a graph, with time on one rectangular axis and stage on the other. Several good graph registers are on the market.[12] All are of the float type. The graph with rectangular coordinates has an advantage over a polar graph in that the time scale is not reduced at low stages, when records are generally most valuable.

[12] W. & L. E. Gurley, Troy, N. Y., seven-day graph.

Julien P. Fries & Sons, Baltimore Md., seven-day graph, sixty-day graph, continuous graph.

Leupold, Voelpel & Co., Portland, Oreg., seven-day graph, continouus graph.

Pressure gages operate by transmitting the pressures of the varying depths of water through a diaphragm to an air chamber connected with the recording device by a flexible tube. They have not given satisfactory service in river gaging because in using them the position of the graph on the record sheet is affected not only by the stage but by any other influence that affects the pressure of the air within the pipe leading from the air chamber to the gage, or by changes in pressure of the air within the pipe, and notably by changes in temperature. The use of a float gage is therefore recommended, although its installation is always more expensive than that of the pressure gage.

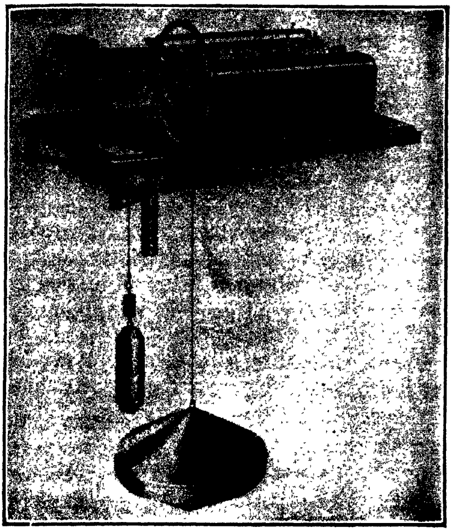


FIG. 492.—A Typical Water Stage Recorder.

489. Operation of Gaging Stations.—The operation of a gaging station will consist of the collection of a record of stage and measurements of discharge, together with such activities as may be needed to

FIG. 493.—Puyallup River, Puyallup, Wash. Recorder Shelter, Cable Tower and Standard Gaging Car

insure the reliability of these two essential factors of the record. If a non-recording gage is used, a reliable gage reader must be employed to read and record the stage once or twice daily. If a 7-day automatic recorder is used, someone must be employed to wind the clock and change the record sheet periodically. If a weight-driven clock and a 60-day or continuous recorder are used, it may be cheapest and most satisfactory to have no one touch the instrument between visits of the engineer. Even under these conditions, however, a local man may well be employed to visit the station at stated intervals to see that the clock is running and to report on the conditions of ice or drift in the vicinity of the gage.

Sufficient measurements of discharge must be made to define accurately the stage-discharge curve. For this purpose each measurement must be made as carefully and accurately as possible. Inaccurate measurements throw doubt on the exact position of the curve and serve no useful purpose. Measurements should, if possible, be made at a constant stage. If made at a changing stage the result will be a sum of partial discharges for the different stages that have occurred during the measurement, but will correspond to no single stage and may have little value in defining the true position of the stage-discharge curve. Measurements should be well distributed over the range of stage, with the greatest number in that portion of the curve where the curvature is changing most rapidly. The upper and lower ends of the station-rating curve are most likely to be poorly defined, on account of the brevity of periods of extreme high and extreme low water, especially the former, and the difficulty in reaching the gaging station during such periods. The lowest point in the control section or, if that cannot be easily found, the lowest point in the cross-section at or near the gage will probably correspond approximately with the stage of no flow, that is, the stage at which the stage-discharge curve will intersect the gage-height axis. This point is extremely valuable in helping to define the position and shape of the lower part of the rating curve.

The stage corresponding to each measurement of discharge should be obtained from the gage from which is made the record of stage that is to be used in computing the record of dischrage, as that gage must be used in determining the stage-discharge curve for the station.

The station having been well rated, occasional careful measurements of discharge are needed to insure the continuous applicability of the rating. Even the most carefully selected and accurately rated stations will sometimes change sufficiently, perhaps temporarily, to disturb the stage-discharge relation. Besides shifting beds and banks, there are local and temporary obstructions, like the lodging of ice or other drift in shoal places or against a projecting rock, that may cut off part of the channel and so bring about a higher stage for a given discharge. In some rivers, growing water plants give much trouble. As the summer advances the plants become larger and cause progressively greater obstruction to the flow and progressively higher stages of river for the same quantity of discharge. With the approach of winter the vegetation disappears, and the stage-discharge relation becomes the same as in previous winters. Such temporary changes in rating will, if

undiscovered, lead to errors of considerable magnitude in the record of discharge.

The gage or gages must be maintained in position. The datum of each gage must be checked at least once a year by Y-level comparison with a bench mark near by. If the gage is inclined, several points on it must be compared with the datum, because of the danger of changes in inclination caused by heaving or settlement of one or more of the supporting posts or piers. If a chain gage is used, the length of the chain must be checked and corrected at least once a year, as an essential part of the maintenance of its datum. The graduations of the gage must be kept clearly legible by washing the face of the gage if it becomes dirty, or by repainting or replacing the face of the gage if the markings become defaced or so faint that they are not easily seen.

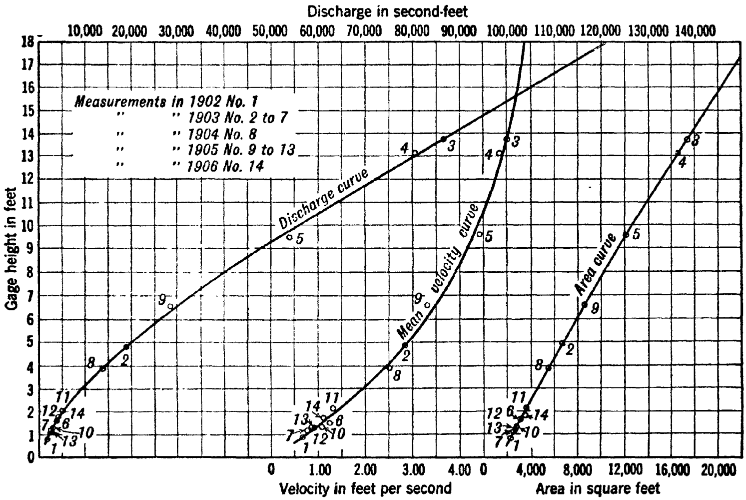


FIG. 494.—Discharge, Mean Velocity, and Area Curves, Potomac River at Point of Rocks, Md.

490. Winter Records.—During periods of partial or complete ice cover, the regular stage-discharge relation is disturbed in varying amounts depending on the extent and thickness of the ice. Such disturbance has the nature of back water and is always manifested by an increase in stage for a given discharge. For those periods it is generally better to correct the records of stage by subtracting the amount of the back-water effect rather than to attempt to prepare a new rating curve for each change in conditions affecting the stage-discharge relation. The amount of the correction for back water must be obtained by occasional current-meter measurements of discharge and by study of the progressive increases in stage with the increases in extent and thickness of the ice, including comparison of changes in stage at gaging stations affected by ice with changes at stations in the same region that are not so affected.

Current-meter measurements of discharge under ice should be made either by the vertical velocity-curve method or by measurements at 0.2 and 0.8 depth. The vertical velocity curve under ice will have a form somewhat different from that of the open channel but will still be parabolic, and therefore the average velocity will be correctly measured by averaging the velocities found at 0.2 and 0.8 of the depth from the bottom of the ice to the bed of the stream.

491. Computation of Daily Discharge.—The conversion of the measurements of discharge and the record of stage, which together constitute the essential field data, into a record of discharge involves the construction of a station-rating curve (Fig. 494) showing the stage-discharge relation applicable to the gage used in obtaining the record of stage, the conversion of this curve into a rating table for convenient and accurate use, the application of the record of stage to the rating table to obtain a record of daily discharge, and the computation, preferably by ordinary computing machines or Crelle's Tables, of records of monthly and annual runoff. Most of these steps involve only care in computation. In the construction of the stage-discharge curve, however, both judgment and care are necessary in adjusting the curve among the plotted discharge measurements, which may be more or less discordant. Judgment must also be used in choosing the methods of determining the mean daily stage to be used in computing the mean daily discharge, or, if the fluctuations are too great and rapid for using an average stage for the day, in deciding on the interval of time, hourly or longer, that may be used without undue sacrifice of accuracy in the computed results.

CHAPTER XXXV

OPERATION OF HYDRO-ELECTRIC PROPERTIES

By William W. Tefft

492. General.—Although the advance in the art of designing and building hydro-electric plants has been very marked during the past ten years, there has not been a parallel advance in the operation of such plants. Whereas, ten or fifteen years ago, the typical hydro-electric plant was an isolated unit, to-day the same plant is a part of a great interconnected system comprising a large number of hydro-electric plants. This has introduced new factors, new advantages and new complications into the problems of operation. Unfortunately, these problems have not, in many cases, received the intelligent attention which they deserve, and the result is that we find plants, which are integral parts of great systems, often operated in much the same manner as the isolated plants of ten or fifteen years ago.

Scientific operation, meaning the intelligent and systematic operation of a hydro-electric property for maximum return, may often increase the gross income 10 per cent or more over what it would have been with indifferent operation. Moreover, for systems where the net income is small compared with the gross income, such an increase in gross income may mean a 30 to 40 per cent increase in net income.

By " operating efficiency " is meant the ratio of the actual energy output for any period to the energy output which might have been obtained for that period with perfect operation. Operating efficiencies of 100 per cent are possible, but 96 per cent is seldom exceeded. Some large systems, improperly operated, have operating efficiencies below 85 per cent.

The usual causes of poor operating efficiencies are:

1. Running the units at gate openings that correspond to poor efficiency;
2. Loss of head due to faulty drawdown of pond;
3. Leakage through units that are shut down;
4. Operating the units at no-load for use as synchronous condensers or for other purposes;
5. Miscellaneous losses of water;
6. Deterioration of the apparatus.

493. Gate Opening.—As shown in Fig. 39, the efficiency of a hydro-electric unit is usually greatest when it is turning out about 85 to 90 per cent of its full gate capacity. At each side of the point of maximum efficiency the efficiency drops off; more rapidly beyond.

If a single plant carried the whole load, the units would have to follow the

demand, regardless of efficiency; but, in large, interconnected systems, where the capacity of a single generating unit is small in comparison with the total system output, it is possible always to have all units on the line running at or very near the point of best efficiency. As the load builds up on the system, additional units can be added, but always at or near the point of best efficiency.

Exceptions to this rule occur as follows:

a. When plants have no pondage and must be operated at an output corresponding exactly to the stream flow.

It is very seldom, however, that the pondage is absolutely negligible. Even for extremely small ponds, if the capacity of several turbines at or near the point of best efficiency does not correspond to the flow, the turbines can, nevertheless, be run at such capacity, and the pond drawn down slightly. Then when one of the units is shut down and the load transferred to other plants, the pond will fill and the operation will be repeated. Such a transfer of load, requiring shifting several times an hour, is common when the capacity of the pond is very small.

b. When, for some good reason, the plant must be operated at the full capacity of all units.

1. When a severe rain is expected and, for some particular plant or plants, it is desired to lower the pond as quickly as possible to retain the increased flow for later use;
2. When, during the period of peak demand, the entire system must be run at full plant capacity to meet the load.

For a plant which is required to carry loads that do not correspond to most efficient output, the maximum possible efficiency is obtained when all units are operating at the same gate opening, provided that all units are similar. This fact is too often overlooked with a resulting loss in economy.

If the units are not similar, the most economical distribution of load among them can be obtained from a study of their respective characteristics, and instructions can then be issued to show the proper distribution for any given load.[1]

494. Drawdown of Pond.—One of the most common faults of operation is the drawing of the ponds, during periods of peak demand, to such an extent that they will not fill during the off-load period. This results in the necessity for operation at unnecessarily reduced heads.

A careful analysis should be made to determine the maximum amount of economical drawdown for each dam. This depends largely on load conditions, pond capacity, and the total head If the drawdown is to be great, the effect of loss of turbine efficiency, due to operating at a head different from that for which the turbine was designed, should not be overlooked in this computation. Where heads are lapped in a series of dams a drawdown can be utilized without any loss of power. This lapping of heads has proved very advantageous in the writer's experience since 1906. When using head increasers such as the Tefft Tube Spillway, care should be exercised not to overload the generators due to the head-increaser effect.

[1] See "Hydro Plant Operating Routine and Tests," Power Plant Engineering, Aug. 1, 1926, p. 860.

Those charged with the operation of hydro-electric plants should make a thorough study of the pondage conditions existing at the various plants in the system. Charts should be prepared showing the kilowatt-hours in each pond for various drawdowns and the number of hours per day that each plant can be operated at various capacities for various average twenty-four-hour flows in the river. The possession of such data is especially important and useful at times of minimum flow when the operating department is using every endeavor to have the hydro-electric plants carry as much of the upper portion of the load curve as possible. A fairly accurate record of the average twenty-four-hour flow in the river at all times is essential. This may be obtained from United States Geological Survey gaging-station records or from the rated discharge of other hydro-electric plants upstream. Accurate reports of the average twenty-four-hour discharge should be obtained daily. The lack of such reports has frequently led to the drawing down of large ponds to such a point that the existing daily stream flow in the off-peak hours was unable to fill them up again and the effective capacity of the plant at minimum flow was thus materially decreased.

Where pondage is sufficient, a weekly schedule of operating the pond may be worked out, such that the pond is at its lowest elevation Saturday noon but fills to crest elevation by Monday morning. Plants without pondage must be operated on the base of the load regardless of the flow in the river, except that plants having no ponds of their own may be able to operate on the pondage of another plant located upstream.

On some rivers the loss of head occasioned by the operation of pondage is quite a material item, but when the heads are lapped there need be no such loss. But even if a material loss of head is obtained by drawing down the ponds, it almost invariably pays to do so, as the hydro-electric energy is thus placed where it is most valuable—in the peak of the load.

As the flow of the river approaches flood stage, the pondages at various plants become less and less effective and the object finally becomes to turn out as many kilowatt-hours as the load curve can take.

495. Turbine-gate Leakage.—The turbine gates are seldom very tight after a few years of operation, especially if allowed to get out of adjustment. For this reason, in plants having long pipe lines, a valve is frequently installed near the turbine.

If the leakage in the turbine gates is excessive, and the turbine is to be shut down for a long period, the valve near the turbine should be closed, or, in the absence of such a valve, the headgates should be closed. This is a troublesome feature of operation and is frequently neglected, with the consequent loss of considerable energy. One objectionable feature of thus unwatering the turbine is the resulting delay in starting up the unit in case of urgent necessity. This is, however, frequently an invalid excuse for allowing the resulting waste of energy, particularly on very large systems.

The turbine-gate leakage can be best determined by observing the tail race with the plant shut down tight but with headgates open. The proper allowable amount of gate leakage is about 1 per cent, although it is often 3 per cent or more in plants that are considered to be well operated, and may be as high as

10 per cent for neglected units. A simple computation of the value of this leakage in terms of power, for any particular plant, will almost invariably show a definite financial profit if the leakage can be decreased. This applies particularly to plants that have installed capacity well above the summer average flow. It should be observed, too, that this loss from leakage is greatest in the low-water time in the summer when it is most important.

496. Operating Turbines at No-load.—One of the frequent sources of loss of energy is the practice of allowing idle units to float on the line at gate opening corresponding to no-load. This practice is most frequent when the units are being utilized as synchronous condensers; but sometimes, although not frequently, they are allowed to float on the line to be ready to pick up load quickly if required. When so used, the turbines might better be vented in order to admit air to the draft tube and break the vacuum. In utilizing hydro-electric generating units for power-factor correction, the Consumers Power Company some years ago vented the draft tubes at the Foote Dam on the Au Sable River in Michigan, thus avoiding the loss of power necessary to drive the turbines as pumps in the draft-tube water.

The generator power factor on a transmission line over 100 miles in length, and with pressures as high as 140,000 volts, generally is leading, owing to the relatively large amount of charging current taken by the line. Even with no load on the receiving end of the line it would require generator capacity of the order of 75 to 100 kv-a. per mile of line simply to energize it. Under this condition, the gate opening on the hydraulic turbines would be practically at the no-load point.[2]

Turbines waste much water when operating at no-load. Whenever possible, it is much better to close the head or turbine gates, empty the draft tube and allow the units to rotate freely. Care should be taken that there is sufficient water being continuously supplied by leakage or other means to the bearings, and also to the periphery of the runner, to prevent it from heating if it should happen to rub in the clearances.

497. Miscellaneous Losses of Water.—In many poorly maintained plants the leakage is considerable through sluice gates, log-chute controls, fishways, dams, conduits and other places. The annual value of such lost energy, in many cases, would permit relatively large sums to be spent for proper maintenance charges.

498. Deterioration of the Apparatus.—Aside from the practice of operating at poor gate efficiency, previously mentioned, perhaps the most common large loss of energy in old plants is the result of deterioration of the apparatus with consequent decrease in efficiency.

[2] The charging kv-a. on a 140,000-volt transmission line at normal voltage can be roughly estimated at 75 to 100 kv-a. per mile. This represents a zero power-factor leading load on the generators, which always exists when the line is operating at normal voltage. When carrying a power load, of course, the reactive component of this load, together with the excitation load of all transformers on the system, neutralizes an equal amount of line charging kv-a., and if these two happen to be equal in amount the resultant generator power factor will be unity.

In order to assist in energizing an unloaded transmission line, reactors are sometimes connected in parallel with the line at the generating end in order to relieve the machines of a portion of the line charging kv-a. Such a reactor neutralizes an equal amount of line charging kv-a.

Tests of hydro-electric plants for efficiency should be made from time to time, perhaps once every two or three years. They do not need to be extensive, but should cover the matter of leakage mainly, as the amount of power that can be absorbed by the rubbing of runners is small. About the only change in efficiency likely to occur in the life of the plant is leakage, on account of improper runner clearances and distortion of cases if they are too light, and cavitation. These will be discovered, by the periodical inspection discussed later in this chapter, before they can have much effect on efficiency. In several systems, pitot tubes have been successfully installed at places of sudden reduction in penstock area. When properly calibrated, these tubes serve as meters to indicate very accurately the amount of water flowing through the units at any time. Such meters are quite valuable for indicating any falling off in efficiency of the units and also may be used to record, continuously and automatically, the water used. A number of such meters have been installed by the New England Power Co. and have been described in "Mechanical Features Affecting Reliable and Economical Operation of Hydro-electric Plants," by E. A. Dow, Spring Meeting of the A. S. M. E., May 18 to 21, 1925.

499. Care of Hydraulic Turbines.—Hydraulic turbines receive less attention, considering the work they do, than any other type of prime movers. Many open-flume turbines operate two or three years without even having the water drained from the flumes or the bearings or gate mechanism inspected. It is frequently found that after this length of time several of the guide vanes or links are broken so that much water is wasted when the unit is shut down. Sometimes the main-shaft bearings become so worn that the runners rub on the wearing rings, which may result in breaking the runner itself, or in setting up serious vibration which loosens the fits between the runner and shaft. It is recommended that open-flume turbines be shut down for inspection once every four to six weeks. This is readily accomplished in a four- or six-unit plant which operates six days a week only, by making an inspection of one unit every Sunday or, in case of continuous load, taking one unit out of service for the necessary time required for this inspection each week. During this inspection the following points should be given attention;

1. Are the water passages, that is, the runner and guide vanes, clear of sticks, timbers, or foreign matter?
2. Check the runner clearances with thickness gages, and adjust the main-shaft bearings to bring the runner to a central position.
3. Inspect the gate mechanism for broken or worn links and check the closing of the gates to see that all gates close tightly.
4. If regulating connections, that is, links, regulating shaft, and bearings, are fitted with grease cups or alemite fittings, force grease in each one of these, as the presence of grease in these bearings will greatly prolong their life, not only by lubricating them, but by preventing rusting.
5. Inspect runner for pitting.
6. Remove all foreign material from the flume.
7. Inspect the drain for leakage and see that everything is in proper condition before refilling the flume.

Probably the principal reason why open-flume turbines receive less attention

than they deserve is because of the difficulty of raising and lowering many of the headgates now in use. Some headgates take two or three men from one to one and one-half hours to open. A motor would do this work in much less time and at much less cost and would pay for itself not only in the saving of labor, but in the more frequent inspection which could be given the unit.

500. Care of Outside-gate-mechanism Turbines.—Turbines having the outside type of gate mechanism, that is, having the regulating mechanism located on the cover plate, are usually provided with grease cups or alemite fittings, to be turned down or greased from one to three times daily, depending on the amount of regulation which the turbine does and upon the quality or condition of the water which is being used by the turbine. Turbines which do very little regulating, that is, which run at practically constant load with the gates blocked, do not require much lubricating, unless the water contains acid, silt, or other material which tends to get into the guide-vane bearings. Probably the best way to determine the amount of lubrication required is, after placing a new unit in operation, to set a certain schedule for lubrication and after, say, sixty days, inspect the guide-vane bearings and see whether they have received proper lubrication; if not, the schedule should be increased. If they are in perfect condition, it may be possible to decrease the schedule. This applies to the upper and lower guide-vane pivots. The pins and links on the shifting ring could get along with less attention, and judgment should be used.

501. Care of Main Turbine Bearings.—Main turbine bearings of the water-lubricated lignum-vitae type should be watched carefully, especially during winter when there is danger of ice clogging the screens. Screens should be of the double type so that one screen will be in service while the other is being cleaned. Screens which may be readily cleaned are recommended, as they will receive attention in proportion to the ease with which they may be cleaned. Sufficient water must be kept flowing through the bearing at all times to keep the entire shell saturated; the best way is to supply an excess amount so that there is a slight amount running out at the top which should be taken away by a drain. In some cases where the stuffing box below the bearing becomes worn and allows a strong suction down through the bearing on account of the vacuum below the runner, it becomes almost impossible to keep water in the bearings. These bearings should receive attention before damage is done. New stuffing boxes or sealing devices should be installed below the bearing to prevent this strong suction. Frequently, lignum-vitae bearings are made adjustable. This type of turbine, whether of the cast-iron, cast-steel, plate-steel, or concrete spiral-cased type, should be drained and inspected once every two or three months, at which time the following points should be looked into:

1. Are the spiral casing, guide vanes and runner passages clear of all foreign material?
2. Do the guide vanes or the runner show any sign of wear, pitting, or erosion? Such action should be reported immediately, and accurate records kept of its progress.

3. With thickness gages, inspect the runner clearances and at the same time inspect the turbine bearing for wear, making such adjustments as are necessary, especially with the lignum-vitae-type bearing.
4. Close the guide vanes and test all vanes for accurate closing to prevent leakage when the unit is shut down.
5. If eccentric pins for adjustments are provided in the gate mechanism, adjust the gates within two or three thousandths for closing. If not so equipped, badly bent gates may be fitted with offset keys.

Turbines of the vertical type which are equipped with oil-lubricated or babbitted main bearings of the vertical type require very little attention for these parts, although they should be watched carefully, and, if they are provided with thermometers, hourly readings of the bearing or oil temperature should be recorded. Horizontal bearings of the ring-oiling type are very reliable and require little inspection, except to see that they do not lose oil, and that the oil level is maintained at the proper height. Vertical-type bearings are usually equipped with some form of oil pump, and on large units these are installed in duplicate, with some automatic device, so that failure of one pump will start the second in operation. Frequent observation is essential, as failure of the pump will ruin the bearing in a few minutes. The level of the oil in the oil reservoir should be carefully watched, frequent inspection being made to make sure that water is not getting into the oil chamber as this will flood the oil out and may ruin the bearing. Babbitted bearings of the vertical type have been in operation for over ten years on some large units with no appreciable wear and very little loss of oil, although it is recommended that the oil be changed and entirely renewed once every year. Sometimes cooling coils are installed in the oil reservoir. In this case the flow of water must be watched carefully to guard against stoppage.

502. Care of Hydro-electric Generators.—It is not the intention here to give any complete instruction regarding the care of generators, but a few important points may be noted. Generators should be shut down and cleaned once every week, as the drawing of air through the generator necessarily deposits a large amount of dust and foreign matter in the windings. If not blown out weekly, this will build up a firm deposit which will eventually clog the ventilating openings of the generator, causing it to heat, and thus eventually decreasing the load which the generator can carry for a safe temperature rise. For this purpose an air jet is desirable, or a suction jet of air, similar to a vacuum cleaner. All ventilating openings should be blown out and all exposed parts wiped off carefully with cloth.

Oil is very injurious to generator windings, and a leaky thrust bearing or steady bearing must be repaired as soon as possible, because if the windings once become saturated with oil they can never be thoroughly cleaned. The oil will cause any dust to stick and this will cake up and make it impossible to remove. Oil will break down the insulating qualities, and sooner or later there will be a short circuit. Many generators have had to be entirely rewound because of oil getting into the windings.

Thrust bearings should be watched, and their temperature recorded hourly, some form of alarm being provided to give warning in case of failure

of the cooling water for the thrust bearing, as this is probably the most important and most heavily loaded of any type of bearing.

503. Effect of Wear on the Efficiency of Turbines.—Many turbines run from ten to twenty years without appreciable wear and without appreciable change in efficiency. On high-head turbines having low specific speed, the clearance spaces around the runner are very important, as even with a new machine and twenty thousandths clearance, the leakage through this clearance area may affect the efficiency from 1 to 3 per cent. With dirty water, this clearance rapidly increases so that, when the clearance has increased to three or four times the original value, the efficiency of the unit may be decreased from 3 to 10 per cent. These clearances should be watched, and a definite limit of maximum clearance set, at which the wearing rings on the runner and in the stationary parts are to be renewed. This applies principally to turbines operating at heads above 250 ft., and more particularly to those operating above 400 ft. The clearances are not so important on the lower heads, and, for units operating below 50 ft., moderate clearances have very little effect. Pitting of the runner is not believed to have a serious effect upon the efficiency of the unit until the pitted area becomes so extensive and the pitting so deep that the shape of the water passage is materially changed; but it is usually found that, before this occurs, the strength of the runner has been seriously affected, and it is desirable either to renew the runner entirely or to build up the pitted area. This question has been covered quite thoroughly on the preceding pages. High-head turbines should be carefully watched at the clearances between the guide-vane ends and the guide case, as, under high heads, if there is sand or silt in the water, this may become rapidly worn, resulting in excessive leakage when the unit is shut down. High-head turbines, when shut down, ought to be provided with some form of a tight valve which should be closed in order to decrease the wear at the sides and contact edges of the guide vanes.

504. Care of Governors.—Practically every hydro-electric unit is now equipped with a governor of the oil-pressure type, and these governors take a large percentage of the operator's time for their maintenance, care, and adjustment. Oil pumps, either of the motor or belt-driven type, must be watched for leakage of joints, temperature of bearings, and noise. Pressure piping from the pump to the pressure tank, from the pressure tank to the governor stand, and from the governor stand to the regulating cylinders, must be watched for leakage; and if it tends to vibrate it should be blocked to hold it steady. If belts are used to drive the flyballs, they must be kept to proper tension, and must be so placed that there will be no jumping of the pilot valve when the lacing passes over either pulley. Regulating cylinders, if located on the cover plate, must be watched for oil leakage along the stuffing boxes, provision being made to drain away any leakage if it occurs; but there is usually not much trouble in maintaining a tight packing on the regulating cylinder. Governor stands, which usually contain a dash-pot, a floating lever, a pilot valve, a synchronizing device, and a load-limiting device, are rather delicate mechanisms, and the adjustments should not be tampered with unless one is thoroughly familiar with the design of the governor. Cleanliness is essential.

The dash-pot oil should be changed yearly, the flyballs lubricated daily or oftener, and the pilot valve frequently inspected to see that it is not worn and does not stick. If a solution with water is used instead of oil, it must be carefully watched so that it does not form a scum around the pilot-valve ports. If a synchronizing motor is provided, belted to the governor, these belts must be watched for tension. The governor apparatus should be kept clean, being wiped off with cloth once a day. In fact, all parts of a generating station should be kept clean, since nothing tends towards careless operation more than a dirty station, and cleaning of the apparatus is usually the best method of inspection, as any leakage, cracks, or other unusual features are readily noted by the careful operator who is cleaning his equipment. This applies not only to governors, but also to generator and turbine equipment.

505. Load Factor.—Assuming that installations have been properly proportioned, plants with ample pondage may, during times of low stream flow, be operated at a low load factor. Frequently such plants, during times of minimum flow, are operated only over the peak hours at full capacity and are completely shut down during the rest of the day.

Any given large system will probably comprise plants with widely-varying load-factor characteristics. There will be plants without any ponds, which must operate on the run of the river at 100 per cent load factor unless the river flow is regulated by other plants with pondage upstream. There will be other plants with ample pondage which may operate on a low load factor, and still other plants which have only a fair amount of pondage. The fitting of the outputs of all these plants with their varying load factors into the load curve in the most advantageous manner requires constant care, study, and good judgment.

506. Interrelation of Steam and Hydro-electric Operation.—Most large power systems include steam plants as well as hydro-electric plants, and the economic operation of the hydro-electric plants is intimately related to the operation of the steam plants. Generally speaking, the steam plants, at times of low water, are operated on the base of the load, and the hydro-electric plants having pondage are operated on the peak. This procedure tends to keep down the investment in steam-generating capacity and also permits the generation of the steam energy on a high load factor, which is much more economical than steam generation on a low load factor.

In the case of systems having very long transmission lines, the contrary practice has been found more economical; that is, the hydro-electric plant is operated on the base, and the steam plants take the peak. This, however, is an unusual method of operating and is due to the fact that the additional investment in the transmission lines required in order to bring the energy into the load center at a low load factor is so very heavy as to be uneconomical when all factors are considered.

506. Venting of Turbines at Low Gate Openings.—The use of venting, to improve the operation of hydro-electric units as synchronous condensers, has already been discussed. The venting of reaction runners at small gate openings increases the power output for a given water discharge, decreases the losses on idle units, permits of obtaining synchronous speed at lower gate

openings—usually 1.5 tenths instead of 2.25 tenths unvented,—and may also be advantageous in reducing noise and vibration. An incidental advantage is that it provides a convenient vacuum cleaning system for the power house.

508. Reliability of Operation of Steam and Hydro-electric Plants.—According to the writer's experience, hydro-electric plants are much more reliable than steam plants.

This conclusion was also reached by the Sub-committee on Mechanical Reliability of Water Power Units of the National Electric Light Association Hydraulic Power Committee.[3]

From extensive and careful study, it has been found that power from a properly designed and constructed hydro-electric plant, delivered over a 100-mile 140,000-volt transmission line, is from two to six times as reliable at the delivery end of the line as is the power from a modern steam plant located at the delivery end of the line and delivering power there. This is a matter not usually appreciated in comparing the relative merits of hydro-electric and steam power.

The inferiority of reliability of the steam plant as compared with the hydro-electric plant and its transmission line is, the writer believes, quite typical of the two methods of generating power. A steam plant with its multiplicity of machines operating at high speeds, high pressures, high temperatures, high metal stresses, and severe and repeated temperature changes, could hardly be expected to be as reliable as the extremely simple hydro-electric units operating at low speed.

In a general way, experience over a period of years indicates that the cities that are entirely dependent upon modern transmission lines, from a modern hydro-electric plant within 200 miles, receive better service than corresponding cities at which a modern steam plant is located.

A hydro-electric plant of three units can be put into operation in an emergency, from no-load (stationary) to full capacity, in less than four minutes, while a steam plant, even with the boilers banked, would require thirty minutes or more, usually one hour.

The matter of hydro-electric reliability extends beyond the hydro-electric plants themselves, and renders the steam plants and also the transmission lines more reliable, inasmuch as an interruption to service in a steam plant can be made up for ten hours or more by the hydro pondage, and likewise an interruption to one transmission line can be made up for by drawing on the stored water and by the increased loading of another transmission line.

509. Operator's Records and Reports.—Readings should be taken every one-half hour throughout the twenty-four, and one log sheet for every twenty-hour hours sent to the main office for study and filing. Every case of unusual trouble should be reported minutely with all relative facts. Any failure of apparatus should be fully covered with a report, and a copy of this report noted on the apparatus card in the designing engineer's office. The study of these reports from time to time gives a complete history of each particular

[3] Page 3 of Report of the Hydraulic Power Committee, National Electric Light Association for 1924–1925, presented at the 48th Convention, San Francisco, June 15, to 19, 1925. (Publication No. 25–15.)

piece of apparatus, as well as collective information about any type of apparatus or the results obtained from apparatus of any particular manufacturer.

Log sheets are designed to suit the requirements of the particular type of plant, and samples are easily obtained from any of the many operating companies. In general, they require the following records at one-half hour intervals:

For Each Generating Unit
- Armature Amperes
- Kilowatts
- Exciter Amperes
- Exciter Volts
- Generator Temperature
- Bearing Temperature
- Governor Opening

Total Indicated Kilowatts
Generator Voltage
Frequency
Power Factor
Head-water Elevation
Tail-water Elevation
Waste Gate or Other Discharge Openings
Transmission Lines and Distribution Lines in Service
Weather Conditions
Temperature of Outside Air
Transformer Temperature

Readings of circuit meters are required at least once a day, and a report on charging of lightning arresters and storage batteries should also be included.

The data from the log sheets are used at the home office to prepare:

1. A continuous system load curve.
2. Total kilowatt-hours for each plant.
3. Total kilowatt-hours for the system.
4. Total kilowatt-hours for each class of users.
5. The operating efficiency of each plant (see Section 492).
6. A check on the draft of the pond and waste water.

In addition to the log sheets, the operators ought to make periodic inspections and reports on the general condition of the plant. This is particularly advisable for new developments, where settlement, leakage or seepage, sloughing of banks, scour, washouts and other sources of trouble are more likely to develop. These inspections are not supposed to be as complete as the official inspections described in the next section, but are sufficient to insure that the operators are vigilant, not only at the power house but throughout the whole development. Daily inspection of certain items is necessary, while for others, weekly and perhaps monthly inspection may be sufficient.

510. Official Inspection and Tests.—No single item about the operation of hydro-electric plants is more important than inspections. In addition to the routine inspections by the operators, previously described, every plant should be thoroughly inspected by a competent man—perhaps two men—once or

twice annually. The best time is in the low-water period in the summer when weather conditions are favorable and the turbines may be shut down, one at a time, without loss of power output.

Besides providing for observations and reports upon conditions throughout the plant, this inspection has the advantage of permitting material to be brought on the ground for repairs that may be required during the succeeding year, although indicated a year in advance, thus avoiding the necessity of a long shut-down of the unit while parts are being shipped. Troubles at hydro-electric stations usually give warning far enough in advance to prevent inconvenience. This does not apply to steam plants.

For description of prevention of ice and trash troubles, see Sections 147, 176, 194 and 225.

The results of an official inspection may be tabulated on a standard form. Printed forms, which are quite elaborate, are used by some power companies. The forms should contain space for remarks on each and every part of the development without exception. The complete list of necessary items is too voluminous to include here. Some forms even include such things as railings, roadways, painting, amount of flood wood in pond, and similar comparatively insignificant items. The actual operating condition of the apparatus is also inspected and reported on, including such items as runner clearances, condition of bearings, packing, temperature, etc. Methods of operation are looked into, and recommendations made. Notes on fire protection, accident prevention, and like matters covered. In fact, each report is intended to cover a minute inspection of every feature pertaining to the successful operation and maintenance of the station.

511. Duties of Operators.—The duties of operators are, in general, to keep the plants in perfect operating condition mechanically; to keep them **perfectly clean**; to operate all spillway gates, generator units, and all other apparatus around the plant under the general orders of the load dispatcher; to pay particular attention to lubrication, and to advise the load dispatcher of all operating conditions each half-hour. Good housekeeping by the operators may be emphasized, as it has a beneficial psychological effect out of all proportion to its cost.

The operators should be particularly schooled for efficiency during emergencies and supplied with detailed emergency instructions, including those for short circuit, fire, floods, and injuries to operators, especially electric shock. Instructions for resuscitation are issued in pamphlet form by the National Electric Light Association.

A complete description of the development, with a set of plans, instructions for operating special pieces of apparatus, sluice, crest and intake gates, and similar apparatus, should be supplied them.

An instruction department for a large power system is well worth while. The duties consist of furnishing each operator with complete and accurate information about the particular plant that he operates, and then making sure that he understands his instructions perfectly, and memorizes them. This is ordinarily accomplished by unexpected examinations held at the plant.

Bogey contests for power output of hydro-electric plants may be conducted

by the instruction department, and a cup awarded to the winning plant. The award may be made periodically and is given to the plant that delivers to its switchboard the highest percentage of its bogey.

512. Dispatching of Loads.—Dispatching of loads is best done orally, by telephone or wired wireless, by one man only. He must have had extensive actual experience in operating. Nothing less will do. On a large system, in case of emergency, part of the dispatching may be delegated by the chief dispatcher to division dispatchers located closer to the plants and consequently having better communication with the plant operators.

Load signaling by change of frequency is possible on some interconnected power systems, but does not seem to represent the best practice, inasmuch as this change of frequency is not allowable in textile or paper mills, and is a departure in some degree from the best service.

In case of emergency, operators who are out of communication with the load dispatcher are guided by fixed rules, such as the following:

Try to lift the line three minutes after trouble;

Try again to lift it three minutes later;

Try again to lift it ten minutes later;

If it still shows trouble, clear it and leave it clear until word can be received from the load dispatcher or patrolman. All patrolmen must know these rules for their own protection.

A load dispatcher ought to be on duty all of every hour of the year—three shifts per day.

His equipment consists of direct telephone or wired wireless communication with every generating or substation operator on the system. Several hydro-electric plants on a river may be on the same telephone circuits, up to perhaps four, each plant with a different ring. In the office of the load dispatcher is a switchboard, at least two telephones—one for reserve—and alongside of the switchboard a miniature diagram of the entire power system, showing on it every generating unit, every switch between the generating units and the low-tension side of all substations, and the connecting transmission lines. All apparatus that is alive in the system shows a red light on the board. These lights are turned on or off by the load dispatcher in accordance with his orders to the operators. This gives a visual representation of the entire system for the use of the dispatcher, and also apprizes the relief dispatcher of conditions when he starts his shift.

The load dispatcher, receiving half-hour reports from all operators on the system, knows fully all the generating, transmission, and weather conditions, and issues orders to the operators as to the amount of load each plant is to carry, and sometimes how it should be carried, although generally each operator has a fixed schedule of operation for any particular load that the load dispatcher may call for. In the case of unusual conditions, however, this schedule may be varied by the load dispatcher, as for an electrical storm. He is able to predict storm conditions accurately by knowing the time, place, and direction and velocity with which the storm is traveling when it first strikes his system. To protect service, it is usual to cut a large system into several sections operating independently, so that the extent of lighting strokes may be minimized.

The drawing down of ponds should receive careful, complete, and scientific analysis by the load dispatcher as to when and how much ponds may best be drawn down. This analysis should include consideration of:

1. Flow in the river.
2. Daily load factor.
3. Load factor over week-end.
4. Loss of power due to decreased head, on account of lowering of pond and raising the tail-water. It often happens that the power loss due to loss of head is more than the power that would be gained by making power-water of what would otherwise be spilled; i.e., the increased total quantity of water passed through the turbines produces less power than as if a lesser quantity of water were used at full head.
5. Area of pond:
   (a) Before drawndown.
   (b) After drawndown.
6. Prospect of rainfall.
7. Emergency.

The load dispatcher needs full information pertaining to these, usually in the form of curves.

513. Cost of Operation.—The approximate cost of operation of hydro-electric stations, transmission lines and substations is covered in Section 472.

514. Operators.—Usually, in well-designed hydro-electric plants of not more than three units, an average of two to five operators is sufficient. The hours are usually long, but the work is pleasant and comfortable, except for the short intervals when the plants "go into trouble."

The modern tendency is to operate small stations by means of automatic or semi-automatic control. Such stations require only periodic visits by an operator and need not average more than one operator to three stations. Sometimes plants as large as 20,000 kw. are operated by remote control.

For generating stations of over 1000-kw. capacity, and under 20,000-kw. capacity, we find that 2½ operators are ample for all ordinary conditions. Occasionally the division superintendent in charge of perhaps five or six plants will employ common labor for making minor repairs and taking care of maintenance and upkeep, the cost of this work being covered in plant costs other than the direct charge for operators. With this exception there appears to be no reason why 2½ operators per plant are not entirely adequate for modern construction designed with a view to minimizing operating charges. Where there is poor arrangement of racks or poor ice protection, additional help might be required on this account. It has been observed in many plants all over the country that inadequacy of design frequently is a heavy tax on operating costs. It is also a matter of observation that in some localities plants are apparently overburdened with cheap labor, which usually is inefficient and expensive regardless of the daily rate of pay. The writer believes that in a well-designed modern plant, efficiently supervised, the rate of 2½ operators per plant—up to 20,000-kw. capacity—will be entirely adequate. Above this capacity it is usually sufficient to increase the number of operators in direct proportion to the capacity of the plant.

515. Operation of Transmission Lines.—Properly designed and constructed transmission lines, for voltages from 40,000 to 220,000 inclusive, need not average more than one patrolman per 50 miles of line, except in mountainous country. Interruptions should not exceed an average of 2.2 per year per 100 miles of line, nor more than 9.4 hours per interruption.

Tree-trimming must be carefully and systematically carried out, no branches being allowed to come closer than 15 ft. from the conductors, allowance being made for annual growth, which, with willows in wet soil, will amount to 6 ft.

516. Operation of Substations.—Substation operators should have about the same qualifications as those for hydro-electric plants. Most small substations, as now constructed, require no permanent operators whatever, only the periodic attention of an inspector being necessary. Manually operated substations of compact arrangement, where all of the equipment is easily reached from the operator's station, require a total of 2½ men. Stations of this type may be of very large capacity without requiring a larger crew.

For stations of more complex arrangement, where equipment is located on several floors of a building, as well as outdoors, or in general where relatively large areas are involved, a crew of 4⅔ or even 7 men may be required.

517. Supervision and Management.—A typical organization arrangement for the production and transmission department of an extensive power system, covering an area 175 miles wide by 200 miles long, with 37 hydro-electric plants and 12 steam plants, and serving 216 cities and towns, might consist of three main divisions:

a. Stenographers and clerks,
b. Statisticians,
c. Load dispatching,

each division reporting to the manager of production and transmission through an assistant manager.

The load-dispatching division might be divided into two or three local sections, each headed by a division superintendent. To emphasize the specialization of the load dispatching, the repairs, purchasing, transportation, etc., are usually handled by others.

518. Rates for Power.—He who is charged with the responsibility of the rate structure of an electric utility must be governed by two fundamental principles:

1. The rates of the company as a whole must produce sufficient revenue to pay operating expenses and taxes, provide for retirements (depreciation), and produce a fair return on the value of the property.
2. The rates for the several classes of customers must be so designed as to distribute the total cost of the service fairly and equitably among the various classes; otherwise discrimination will result.

We are concerned here primarily with the question of power rates. It must be assumed that the above-named principles have been satisfied. The limited space available forbids any discussion of the broad questions of value, fair return, and discrimination. These are largely questions of law. Only thorough study of the decisions of courts and commissions will give an understanding of

these matters and the principles involved. "The Economics of Public Utilities," by L. R. Nash, and "Guiding Principles of Public Service Regulation," by Henry C. Spurr, are recommended to those who wish to study these subjects.

Customers should be classified and different rates designed for the various classes. Differences in rates are proper where they are based on differences in the characteristics of the use of service, and are fairly proportional to the differences in the cost of furnishing the service. The law does not require, nor is it possible to make, rates which will do exact justice to individual customers. Accuracy must often be sacrificed to convenience of classification.

Rates for the larger power and lighting customers must compete with the cost of other sources of energy. The fundamental advantages of utility power as compared with isolated plants will enable the rate-maker to meet the competition successfully in the great majority of cases. He must, however, remember that competition does not justify him in making discriminatory rates.

It is contended that it is sometimes proper to design rates based upon the so-called "additional cost basis." Such a rate contemplates that the customer shall pay the added costs, including fixed charges on the additional investment required to serve him. This basis of rate-making may be proper when the business can be secured on no other basis, when the rate will not only cover the additional cost, but will also bring something for return on the investment as a whole, and when the rate so made will not discriminate against other customers.

Those costs which are fixed, which are dependent upon investment and not proportional to output, are essentially large for electric utilities. This is a matter of particular importance where all or a large part of the energy is generated by water power. Load factor is, therefore, of the greatest importance in power rates. It is the element which the rate-maker must always consider.

New uses for electricity are being developed continually. Existing customers often change their characteristics of use of power. Improvements in equipment and operating methods are constantly bringing about greater efficiencies. Changing price levels materially affect investment or operating costs, or both. It is obvious that flexibility is of tremendous importance in a successful rate structure. Flexible rate structures can be adapted readily to meet changing conditions.

The power customers of a given utility will vary in size from one to several thousands of horsepower. Some are irregular or intermittent users. The demands of others are remarkably regular and uniform over the year. No one rate will successfully handle all of the great variety of users of power service. There must be several rates of different type and structure, all coordinated into a well-balanced, flexible set of schedules.

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